PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 59 FEBRUARY, 1933 No. 2

TECHNICAL PAPERS
DISCUSSIONS
APPLICATIONS FOR ADMISSION AND TRANSFER

Published monthly, except June and July, at 99–129 North Broadway, Albany, N. Y., by the American Society of Civil Engineers, Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, December 28, 1931, at the Post Office at Albany, N. Y., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy.

CURRENT PAPERS AND DISCUSSIONS

	Published	Discussion closes
Engineering Features of the Illinois Waterway. *Walter M. Smith*	Oct., 1931	
Discussion	Feb., Mar., 1932	Closed
Public Supervision of Dams: A Symposium	Jan., 1932	
Discussion. (Authors' closures)	Mar., Apr., May, Aug., 1932, Feb., 1933	Closed
Determination of Principal Stresses in Buttresses and Gravity Dams. *W. H. Holmes*	Jan., 1932	
Discussion	May, Aug., Oct., Dec., 1932	Closed
Stresses in Reinforced Concrete Due to Volume Changes. *C. P. Vetter*	Feb., 1932	
Discussion	May, Sept., 1932	Closed
Stresses in Inclined Arches of Multiple-Arch Dams. *George E. Goodall and Ivan M. Nelidov*	Mar., 1932	
Discussion. (Authors' closures)	Oct. 1932, Feb., 1933	Closed
The Compensated Arch Dam. *A. V. Karpov*	Apr., 1932	
Discussion	Oct., 1932	Closed
Development of Hydro-Electric Power as an Aid in Irrigation. *Charles C. Cragin*	Apr., 1932	
Discussion	Oct., 1932	Closed
Improvement of Hudson River by Narrowing the Navigable Fairway. *Lynne J. Bevan*	May, 1932	
Discussion	Oct., 1932	Closed
Application of Duration Curves to Hydro-Electric Studies. *G. H. Hickox and G. O. Wessenauer*	May, 1932	
Discussion	Oct., Nov., 1932, Jan., 1933	Closed
George Washington Bridge: General Conception and Development of Design. *O. H. Ammann*	Aug., 1932	
Discussion	Nov., 1932, Jan., Feb., 1933	Feb., 19
Forests and Stream Flow. *W. G. Hoyt and H. C. Troxell*	Aug., 1932	
Discussion	Sept., Nov., Dec., 1932, Feb., 1933	Feb., 19
Tests of Riveted and Welded Steel Columns. *Willis A. Slater and M. O. Fuller*	Sept., 1932	
Discussion	Jan., Feb., 1933	Mar., 19
The Martinez-Benicia Bridge. *W. H. Kirkbride*	Sept., 1932	
Discussion	Nov., 1932, Feb., 1933	Mar., 19
Pre-Qualification of Contractors. *C. J. Tilden*	Sept., 1932	
Discussion	Nov., 1932, Jan., 1933	Mar., 19
Tests for Hydraulic-Fill Dams. *Harry H. Hatch*	Oct. 1932	
Discussion	Jan., Feb., 1933	Mar., 19
George Washington Bridge: Organization, Construction Procedure, and Contract Provisions. *Edward W. Stearns*	Oct., 1932	
Discussion	Dec., 1932	Mar., 19
Foundation Treatment at Rodriguez Dam. *Charles P. Williams*	Oct., 1932	Mar., 19
A History of the Development of Wooden Bridges. *Robert Fletcher and J. P. Snow*	Nov., 1932	
Discussion	Jan., Feb., 1933	Mar., 19
Distribution of Shear in Welded Connections. *Henry W. Troelsch*	Nov. 1932	
Discussion	Feb., 1933	Mar., 19
Work of Rivets in Riveted Joints. *A. Hrennikoff*	Nov., 1932	Mar., 19
Study of Stilling-Basin Design. *C. Maxwell Stanley*	Nov., 1932	Mar., 19
A Problem of Soil in Transportation in the Colorado River. *S. L. Rothery*	Dec., 1932	Mar., 19
George Washington Bridge: Design of Superstructure. *Allston Dana, Aksel Andersen, and George M. Rapp*	Dec., 1932	Mar., 19
George Washington Bridge: Design of the Towers. *Leon S. Moisseiff*	Dec., 1932	Mar., 19
Model Law for Motion of Salt Water Through Fresh. *Morrough P. O'Brien and John Cherno*	Dec., 1932	Mar., 19
Wind Stress Analysis Simplified. *L. E. Grinter*	Jan., 1933	Apr., 19
George Washington Bridge: Construction of Substructure. *Montgomery B. Case*	Jan., 1933	Apr., 19
George Washington Bridge: Construction of Steel Superstructure. *E. W. Bowden and H. R. Seely*	Jan., 1933	Apr., 19

CONTENTS FOR FEBRUARY, 1933

PAPERS

DISCUSSIONS

CONTENTS FOR FEBRUARY, 1933 (*Continued*)

For Index to all Papers, the discussion of which is current in* PROCEEDINGS, *see page 2

MEMBERSHIP

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

EVAPORATION FROM WATER SURFACES
A SYMPOSIUM

NOTE.—Written discussion on this Symposium will be closed in May, 1933, *Proceedings*.

FOREWORD

These two papers and the recommended list of standard equipment constitute the Final Report of the Special Committee on Irrigation Hydraulics on the subject of evaporation. The Symposium has been developed since the submission of the work done by Mr. Rohwer, which was accepted for publication as a separate paper. The official report of the Sub-Committee on Evaporation, covering only the subject of standard equipment, was next added; and, finally, Mr. Follansbee's contribution was accepted to round out the entire subject.

The Committee has had as one of its subjects for investigation that of Evaporation from Reservoirs. One of the primary objects of that work has been to find a suitable set of coefficients by which the great variety of existing data on evaporation could be corrected, to the end that evaporation from a large water surface could be expressed in terms of that from the various types of pans in use.

The first objective has been fulfilled in the paper by Mr. Rohwer and the second by the report of the Sub-Committee on Evaporation. Mr. Follansbee's paper serves the purpose of applying the findings regarding pan evaporation to reservoirs. A great mass of detailed information, comprising 200 or more tables, has been placed on file by Mr. Follansbee, at Engineering Societies Library, 29 West 39th Street, New York, N. Y. These records contain data pertaining to evaporation from reservoirs all over the United States and in many other parts of the world.

The Special Committee on Irrigation Hydraulics has approved the report of the Sub-Committee on Evaporation unanimously and, further, recommends that these standards be followed wherever new evaporation stations are to be installed or old ones re-equipped.

By the Special Committee on Irrigation Hydraulics,

D. C. Henny, *Chairman,*
J. C. Stevens, *Secretary,*
B. A. Etcheverry
George W. Hawley
Ivan E. Houk
Morrough P. O'Brien
R. L. Parshall
J. L. Savage
Fred C. Scobey
I. C. Steele
Franklin Thomas

January 18, 1933.

EVAPORATION FROM DIFFERENT TYPES OF PANS

By Carl Rohwer,[1] Assoc. M. Am. Soc. C. E.

Synopsis

The rate of evaporation from different types of pans varies widely and before the evaporation data can be used in estimating the loss from large bodies of water, it is necessary to know the ratio between the evaporation from the type of pan used and that from a large water surface, or the relation between the evaporation from the pan and some pan for which the ratio of the evaporation to that from a large water surface is known.

A summary of the results of the available records, where a comparison has been made between the evaporation from different types of pans or between the evaporation from a pan and a large water surface under similar conditions, is given in this paper, together with recommendations as to the best types of pan to use under different conditions, and the procedure to be followed in taking the observations.

Introduction

Available records of evaporation from pans of different types, or from pans and large water surfaces under similar conditions, are either scattered through many publications or are buried among the unpublished records of various organizations. The writer has assembled such data and re-arranged them for presentation in tabular form. Credit is given in every instance to the publication or agency from which the record was obtained. The tables are published as an Appendix to the paper.

Comparative Results

The records of the comparisons between the evaporation from many types of pans were taken (see Appendix), but particular attention was given to the pans most commonly used, such as the United States Weather Bureau Class *A* land pan and floating pan, the Colorado sunken pan, the United States Geological Survey floating pan, and the United States Bureau of Plant Industry sunken pan. Comparisons were also made, where records had been taken, between the evaporation from these pans and from large bodies of water. In addition to the record of the evaporation, all meteorological data pertaining to each tank were included. Where possible the data at the station were used, but when no records had been kept, the data were taken from the records of the nearest Weather Bureau Station.

[1] Assoc. Irrig. Engr., Colorado Experiment Station, Fort Collins, Colo.

All the records have been summarized and only the mean data are included in this paper. In some cases merely the means for the entire period of the observations are given, but where sufficient records are available the monthly or the daily means are reported. The daily means are given only in those cases where intensive studies of evaporation were made for short periods, and where the observations were taken at frequent intervals during the day.

None of the records has been eliminated, nor have any observations been discarded, except where the original records indicate that the observations are in error. It was assumed that the observations were correct, except as noted, and that differences in the results were due to variations in the setting or the conditions under which the pans were exposed. Consideration was given, however, to the fact that the records from floating pans are less reliable than those from land pans on account of the difficulty in making accurate readings of the evaporation loss, and also because during wind storms water frequently splashes into the pan from the reservoir, or splashes out of the pan when there is a pronounced swell after a storm.

The comparisons of evaporation from pans and from large water surfaces are few and the results are uncertain because of the difficulty in segregating the evaporation from the other losses. Experiments on the evaporation from pans of different sizes, however, show that the size of the pan has a proportionately less effect on the evaporation as the size of pan increases. This fact is used as the basis for determining the relation between the evaporation from pans and from large water surfaces. The comparative data on the evaporation from reservoirs and from pans similarly exposed are presented to show how closely the facts are in accord with the theory, and also to give some idea as to the variation that may be expected in the results.

In reporting the data all the known details concerning the setting of the pans and the conditions under which they operated, are included. Unfortunately, many details are frequently lacking in the reported data. Under these circumstances, if there are discrepancies in the results the reasons for them can only be conjectured.

The tables giving these data are arranged according to the types of pans. All tables showing the comparison of one type of pan with various other types are grouped together. From the mean values taken from these tables, a summary of all the results in which a comparison with any of the standard pans was possible, was prepared. These mean ratios are not the means of the monthly ratios, but are the means of the ratios of the total evaporation losses from the pans.

The experiments[2] made by the late R. B. Sleight, Assoc. M. Am. Soc. C. E., at Denver, Colo., during 1915, 1916, and 1917, cover a wide range of conditions, and the results obtained are uniformly consistent. In using the ratios for the floating tanks, however, it should be kept in mind that the floating pans were not placed at the same point as the land pans and that results similar to those obtained elsewhere can not be expected. The obser-

[2] *Journal of Agricultural Research,* Vol. X, No. 5, July 30, 1917.

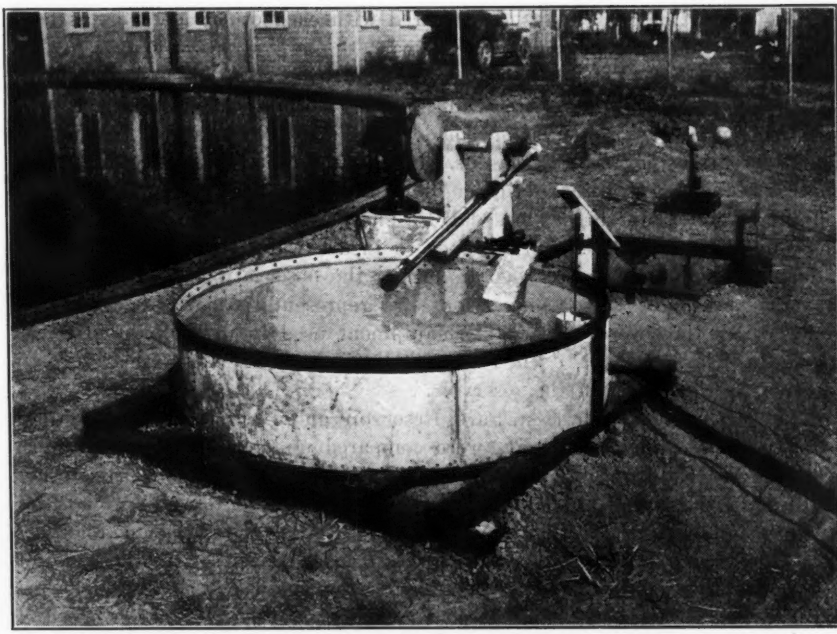


FIG. 1.—U. S. WEATHER BUREAU CLASS *A* LAND PAN, COLORADO SUNKEN PAN, AND 86-FOOT CIRCULAR RESERVOIR AND AUXILIARY EQUIPMENT AT FORT COLLINS, COLO.

FIG. 2.—ARRANGEMENT OF EQUIPMENT FOR CONDUCTING EXPERIMENTS ON THE 85-FOOT CIRCULAR RESERVOIR AND VARIOUS STANDARD PANS AT FORT COLLINS, COLO.

vations of floating pans may be compared with each other, however, as they were all subjected to the same conditions.

The comparisons of the evaporation from different types of pans with that from the 85-ft circular reservoir at Fort Collins, Colo., are the results of a large number of observations made under carefully observed conditions. The observations cover three seasons. During the first two seasons, the readings were taken four times daily at 6-hr intervals, and during the last season they were taken three times daily. The 85-ft reservoir used in making the comparison was lined with copper to insure water-tightness. Carefully calibrated instruments were used in taking the observations. It is believed that the results of these comparisons are representative of the conditions under which they were taken. The equipment used in making the study is shown in Figs. 1 and 2. In Fig. 1, the Geological Survey floating pan is shown in the center of the reservoir.

The evaporation from East Park Reservoir and from the various evaporation pans at Stonyford, Calif., were compared under conditions that were favorable for obtaining reliable results. East Park Reservoir is in a practically water-tight basin, and during the period of the tests there was no precipitation and practically no inflow or outflow. Evaporation readings were taken twice daily. The results obtained are in accord with those found at Fort Collins and Denver, except in the case of the floating pan at Denver, and as previously stated, the land and floating pans at Denver were some distance apart. Figs. 3 and 4 show the evaporation pans at East Park Lake.

The ratio of the evaporation from the Weather Bureau Class *A* land pan and the Colorado sunken pan to the evaporation from the 12-ft circular sunken pan at Denver, the 85-ft circular reservoir at Fort Collins, and the 1 800-acre reservoir at Stonyford, are almost identical. Therefore, it is evident that between these limits (surfaces of 113 sq ft and 1 800 acres), there is no decrease in the evaporation per unit area as the size of the water surface increases. These conclusions are confirmed by experiments at Milford, Utah, on the evaporation from a Weather Bureau Class *A* land pan and a 12-ft circular sunken pan. In view of the fact that these observations were made at widely separated points, it may be assumed that these ratios are constant. If these conclusions are correct, it is possible to compute the evaporation from a lake or reservoir when the evaporation from any type of pan for which the ratio has been determined, is known.

Other Reservoir Evaporation Comparisons

A comparison of the evaporation from a buried pan and the total loss from Newell Reservoir, Alberta, Canada,[3] by the Department of Natural Resources of the Canadian Pacific Railroad for the period, 1919 to 1925, shows that on the average the total loss from the reservoir is 95% of that from the land pan. The season ratios, however, vary from a minimum of 77% to a maximum of 116 per cent. This large variation is probably due to the fact that the loss from the reservoir was determined from the seasonal inflow, outflow, precipitation, and change in elevation. Errors in any of

[3] *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 350.

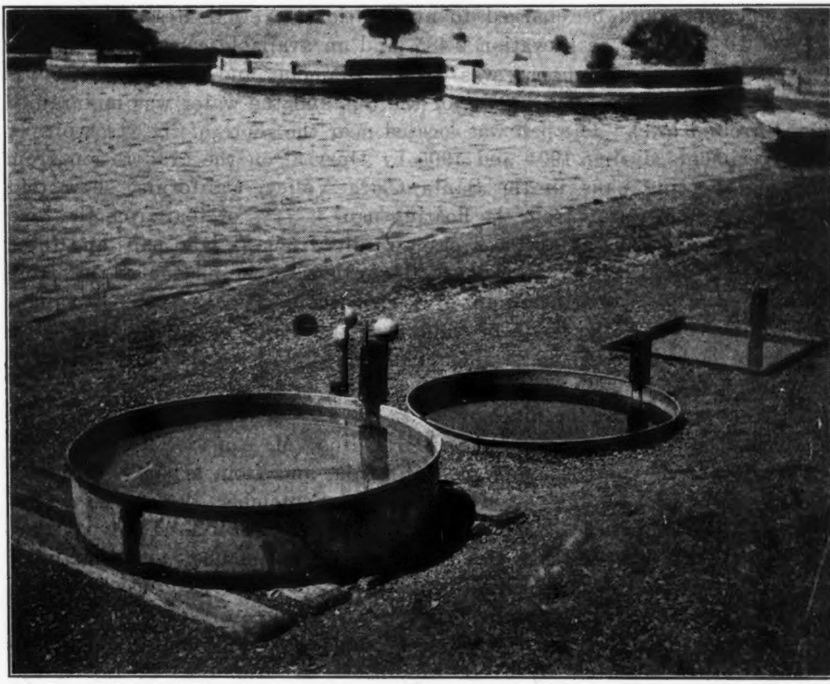


FIG. 3.—U. S. WEATHER BUREAU CLASS *A* LAND PAN, 4-FOOT CIRCULAR SUNKEN PAN, AND COLORADO PAN, AT EAST PARK RESERVOIR, STONYFORD, CALIF.

FIG. 4.—U. S. WEATHER BUREAU CLASS *A* LAND PAN USED AS A FLOATING PAN AT EAST PARK RESERVOIR, STONYFORD, CALIF.

these factors would be charged to evaporation. Newell Reservoir has an area of 16 430 acres at Elevation 2 485, and an available capacity of 189 000 acre-ft. The evaporation pan was 4 ft in diameter and 2 ft deep. It was sunk in the ground to within 2 in. of the top, and the water was maintained at the ground level. The pan was located near the shore of the reservoir.

Experiments during 1904 and 1905 by Duryea[4] on the evaporation from land and floating pans in the Santa Clara Valley, California, show that the average evaporation from the floating pans is 71% of that from the land pan. This value is based on the average results from six land pans and three floating pans divided into two groups. The values of the relation varied from a minimum of 55% to a maximum of 85 per cent. All pans were 3 ft square and 12 in. deep and the depth of water was approximately 10 in. The land pans were embedded in the ground and banked with earth nearly to their rims. The floating pans were floated on the Laguna Seca and, apparently, were freely exposed to the air.

Experiments were reported by S. T. Harding,[5] M. Am. Soc. C. E., on the evaporation from Lake Elsinore, in California, and from a pan 3 ft square floating in the lake for the period from May to September, 1916. The results show that the evaporation from the lake is 84% of that from the pan, but that the relation by months varies from a minimum of 63% to a maximum of 114 per cent. The evaporation from the lake was determined from the record of the inflow, outflow, precipitation, and change of stage. The high-water area of Lake Elsinore is 5 500 acres and the elevation, 1 261 ft.

Experiments reported[6] by J. B. Lippincott, M. Am. Soc. C. E., on the evaporation from Lake Hodges and Cuyamaca Lake, and the evaporation from floating pans and land pans are contradictory. The evaporation from Lake Hodges (area, 1 317 acres, and elevation, 315 ft), during six months from June, 1919, to October, 1921, when there was no draft from the lake, was 96% of the evaporation from a sunken pan 3 ft square and 18 in. deep, sunk with the rim flush with the ground and filled with water to within 4 in. of the top. The evaporation from Cuyamaca Lake (area, 978 acres, and elevation, 4 600 ft) during November, 1916, and January, 1919, when there was no draft from the lake, was 108% of that from a floating pan 3 ft square and 18 in. deep, filled with water to within 4 in. of the top. Later, the pan was sunk in the ground and under these conditions, during November, 1921, and June, 1922, the evaporation from the lake was 119% of the pan evaporation. Cuyamaca Lake is surrounded by a large swamp area, and the vegetation in this area may have increased the loss, due to the transpiration from the plants.

Choice of Type of Evaporation Pan

The type of pan chosen for evaporation observations will depend on the purpose of the experiment and on a knowledge of the information available for the different types of evaporation pans. Although a pan may be desir-

[4] *Engineering News*, Vol. 67, pp. 380–383.

[5] San Jacinto River Hydrographic Investigations, 1922, Div. of Water Rights, State of California, *Bulletin 9*, State Dept. of Eng.

[6] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 356–359.

able from several standpoints, it should not be chosen unless the relation of the evaporation from the pan to other types of pans and to large water surfaces is thoroughly established.

The types of pans most commonly used in the United States are: (1) The U. S. Weather Bureau Class *A* land pan shown in Figs. 2, 3, and 4; (2) the U. S. Bureau of Plant Industry sunken pan; (3) the Colorado sunken pan (see Figs. 2 and 3); (4) the U. S. Geological Survey[7] floating pan; and (5) the U. S. Weather Bureau floating pan. Each of these pans is built according to definite specifications and has certain advantages and disadvantages. For convenience, these types will be referred to by number in this paper.

Pan (1).—The Class A Land Pan of the United States Weather Bureau.— The Class *A* land pan is used more than any other type, and for general evaporation studies is probably the most satisfactory. This is the standard pan of the U. S. Weather Bureau and is the type used at all its evaporation stations. Other agencies also frequently install this type of pan and follow the Weather Bureau procedure in taking the observations. As shown in Fig. 5, this pan is 4 ft in diameter and 10 in. deep. It is made of 22-gauge galvanized iron and is supported on a grillage of timbers so that the bottom of the tank is 6 in. above the original ground surface. The pan is filled with water to within 2 in. of the top and is refilled as soon as the water has dropped 1 in. below this elevation. A special micrometer hook-gauge is used to measure the evaporation. This gauge is located in a stilling-well which acts as a support for the gauge. The auxiliary equipment consists of an anemometer, a rain gauge, "maximum-and-minimum" thermometers, and an instrument shelter. The readings are taken twice daily at approximately 7:00 A. M. and 7:00 P. M.

The pan is simple in design and easy to operate. Being above the ground, it does not blow full of water in wind storms. The water does not splash into it and the snow does not drift into it when rain or snow is falling. The water in the pan is fully exposed to the air, and follows the air temperatures quite closely. As a result, the evaporation from it changes more quickly with the changes in temperature than from other types of pan. In some experimental work, this is a desirable feature, but the rapid change in the temperature makes it difficult to obtain the true mean temperature of the water.

From the standpoint of making comparisons with known data on evaporation, this type of pan is probably best adapted to the study of evaporation from water surfaces, because a large number of these pans have been installed in the United States, and many observational data obtained under similar conditions with a uniform procedure, are available from them. It is to be regretted that relatively few of these pans have been installed in the eastern part of the United States, and, unfortunately, all the data necessary for making a comparison between evaporation records at different stations are

[7] In a letter dated December 11, 1929, N. C. Grover, M. Am. Soc. C. E., Chief Hydraulic Engineer of the U. S. Geological Survey, states that there is no official U. S. Geological Survey floating pan. The pan by that name was designed by E. F. Kriegsman, Assoc. M. Am. Soc. C. E.

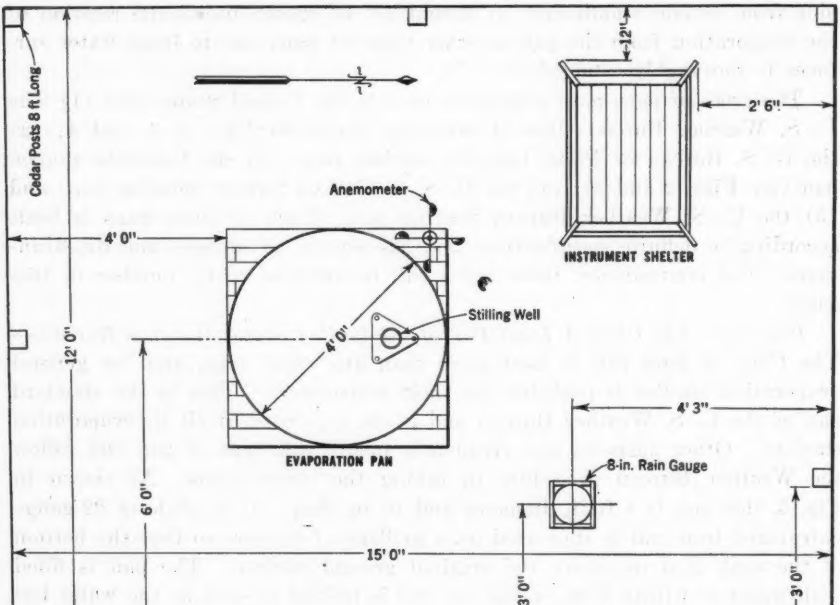


FIG. 5.—PLAN OF U. S. WEATHER BUREAU CLASS A, LAND PAN (TYPE (1)).

not observed by the Weather Bureau. The missing information, however, can usually be obtained from records of near-by Weather Bureau Stations.

Another criticism of this type is that the rate of evaporation is much higher than that from a large water surface under similar conditions, due to the fact that the pan is more fully exposed to the air than the reservoir. For this reason a large correction must be made in determining the reservoir

evaporation from the pan record. This factor has been determined, however, under a wide variety of conditions, with very consistent results. Comparisons indicate that a factor of 0.69 or 0.70 should be used in converting the evaporation from the pan to that from the reservoir.

Pan (2).—United States Bureau of Plant Industry Sunken Pan.—The pan shown in Fig. 6 is also quite satisfactory. It is 6 ft in diameter and

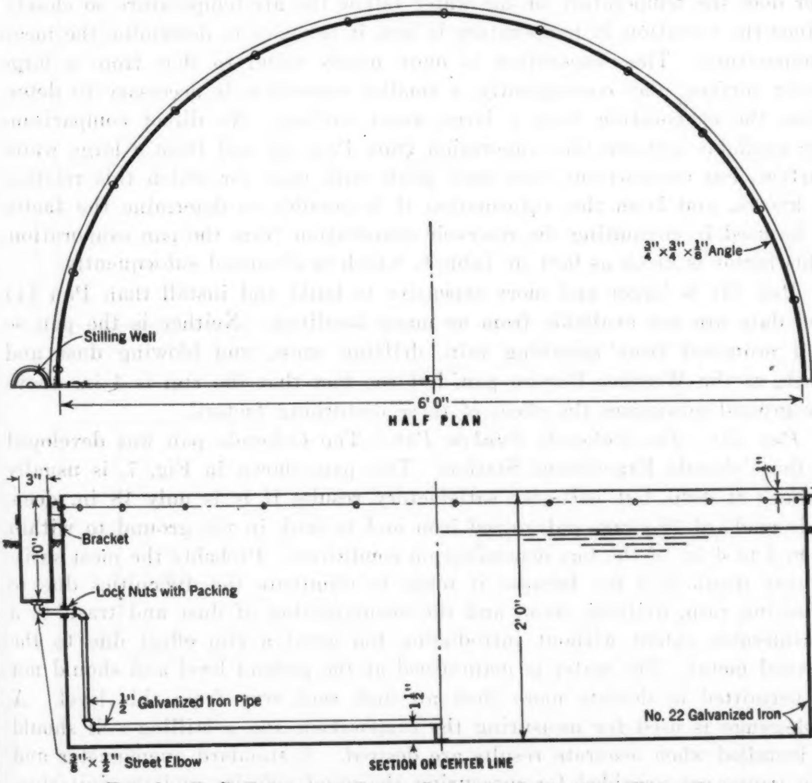


FIG. 6.—U. S. BUREAU OF PLANT INDUSTRY LAND PAN (TYPE (2)).

2 ft. deep, is made of 22-gauge galvanized iron, and is sunk in the ground so that 4 in. of the rim projects above the ground surface. The water surface is maintained at approximately the elevation of the surrounding ground, and whenever it deviates more than ½ in. from the level, due to either evaporation or precipitation, it is brought back to the standard elevation by adding or removing water. The evaporation from the pan is measured by a special micrometer point-gauge, placed on the stilling-well, which is attached to the outside of the pan. A standard rain gauge, anemometer, and instrument shelter of the Weather Bureau type are a part of each station. Maximum-and-minimum thermometers and a psychrometer are provided for determining the temperature and the relative humidity. The readings are taken daily.

The Bureau of Plant Industry maintains an evaporation pan of this type at each of its dry land stations in the western half of the United States. Evaporation records, including complete meteorological data, are available at each of the stations up to and including 1916. After this date the water temperature records are lacking. The evaporation from this pan does not vary through as wide a range as that from Pan (1), nor does the temperature of the water follow the air temperature so closely. Since the variation in temperature is less, it is easier to determine the mean temperature. The evaporation is more nearly equal to that from a large water surface, and, consequently, a smaller correction is necessary to determine the evaporation from a large water surface. No direct comparisons are available between the evaporation from Pan (2) and from a large water surface, but comparisons have been made with pans for which this relation is known, and from this information it is possible to determine the factor to be used in computing the reservoir evaporation from the pan evaporation. This factor is given as 0.94 in Table 6, which is discussed subsequently.

Pan (2) is larger and more expensive to build and install than Pan (1) and data are not available from as many localities. Neither is the pan so well protected from splashing rain, drifting snow, and blowing dust and trash, as the Weather Bureau pan, but the fact that the rim is 4 in. above the ground minimizes the effect of these disturbing factors.

Pan (3).—The Colorado Sunken Pan.—The Colorado pan was developed at the Colorado Experiment Station. This pan, shown in Fig. 7, is usually made 3 ft deep, but will give satisfactory results if it is only 18 in. deep. It is made of 18-gauge galvanized iron and is sunk in the ground to within from 2 to 6 in. of the top, depending on conditions. Probably the most satisfactory depth is 4 in., because it tends to eliminate the difficulties due to splashing rain, drifting snow, and the accumulation of dust and trash to a considerable extent without introducing too great a rim effect due to the exposed metal. The water is maintained at the ground level and should not be permitted to deviate more than an inch each way from this level. A hook-gauge is used for measuring the evaporation and a stilling-well should be installed when accurate results are desired. A standard anemometer and rain gauge are provided for measuring the wind velocity and precipitation, and maximum-and-minimum thermometers for the temperature. A sling psychrometer is used to determine the humidity. Readings on the pan are taken twice daily.

The Colorado pan has not been used as extensively as Pans (1) and (2). It has advantages, however, which warrant a wider use of this type. It is cheap and easy to build and install. Due to the fact that the pan is sunk in the ground, the water temperature lags behind the air temperature. In this respect it resembles a large water surface more nearly than Type (1), and is similar to Pan (2). For this reason the mean temperature of the water is easy to determine. The evaporation from the Colorado pan is almost identical with that from a floating pan of the same size and shape, exposed under the same meteorological conditions, and is much easier to maintain.

The correction factor, 0.78 (in Table 6), for computing the equivalent evaporation from a reservoir is between that for Pans (1) and (2). It has been determined under a wide range of conditions with uniformly consistent results.

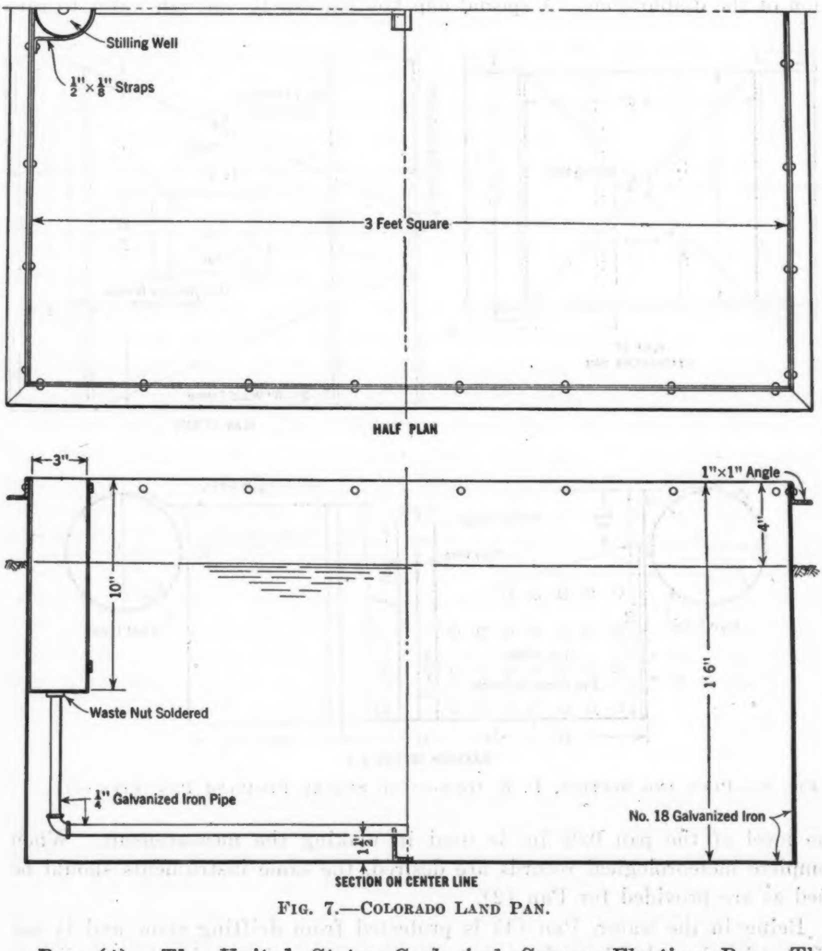


FIG. 7.—COLORADO LAND PAN.

Pan (4).—The United States Geological Survey Floating Pan.—The evaporation pan in Fig. 8, was designed for use in measuring the loss from lakes and reservoirs because it was thought that evaporation from a floating pan would be identical with that from a reservoir, but the experimental results have not substantiated this conclusion. This pan is made of 18-gauge galvanized iron and is supported by two cylindrical metal tubes, so that it floats in the water with 3 in. of the rim above the surface. The water inside the tank is kept at the same depth as that outside. Surging in the pan is reduced by two diagonal diaphragms beneath the water surface, which are

perforated with 1-in. holes as shown. In order to protect it from the waves, the pan is surrounded by a raft supported by barrels to keep it from sinking when water-logged. Evaporation is determined by measuring the water required to bring the level to a fixed point which is attached to the intersection of the diaphragms. A special cup holding exactly enough water to raise

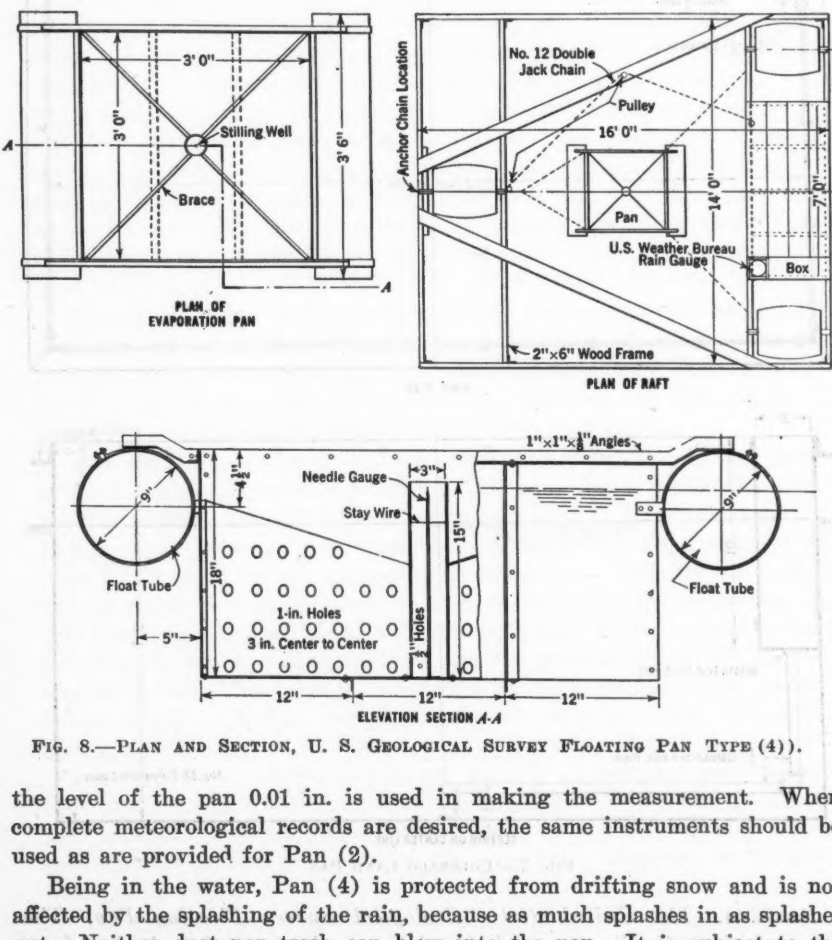


FIG. 8.—PLAN AND SECTION, U. S. GEOLOGICAL SURVEY FLOATING PAN TYPE (4)).

the level of the pan 0.01 in. is used in making the measurement. When complete meteorological records are desired, the same instruments should be used as are provided for Pan (2).

Being in the water, Pan (4) is protected from drifting snow and is not affected by the splashing of the rain, because as much splashes in as splashes out. Neither dust nor trash can blow into the pan. It is subject to the same conditions as those that occur in the lake or reservoir, and, consequently, the factor for computing the equivalent evaporation from the large water surface should be nearer unity than the factors for the land pans. The experimental results are inconsistent; however, the mean value of the factor is 0.77. The evaporation from the floating pan is almost identical with that from a similar sunken pan (see Table 3(*a*) and Table 6). Since, in comparison with the land pan (Type (1)), Pan (4) is more expensive to construct, more diffi-

cult to maintain, and is constantly subject to the danger of being splashed full of water from outside waves or being partly emptied by water splashing out of the tank, due to the rolling of the pan, this type is losing favor rapidly and is being replaced by some type of land pan.

Pan (5).—United States Weather Bureau Floating Pan.—Pans of the Weather Bureau type, as shown in Fig. 4, are sometimes used as floating pans. They have all the advantages and disadvantages of Pan (4), except that they roll more when there is a swell because of their greater size. This condition has been overcome to some extent by building cellular baffles in the pan below the water surface. Evaporation is almost identical with that from a buried pan of similar shape (see Table 1). The constant for converting the pan evaporation to reservoir evaporation has been established as 0.78 (Table 6). Equipment similar to that used at stations of the Bureau of Plant Industry is recommended.

From the foregoing discussion and from the experimental results, Pan (1) seems to be best adapted to the study of ordinary evaporation problems. For special problems, some of the other types of pans may give better results and, under these conditions, the merits of the different pans for the special problem should be investigated. The evaporation from Pan (2) probably approaches that from a large body of water more nearly than any of the other standard pans, but the factor for converting the pan evaporation to the reservoir evaporation is not known as definitely as for some of the other pans. Although this factor has been determined for the Colorado pan (Type (3)) under a wide range of conditions, this type has been used to only a limited extent in evaporation investigations, and, consequently, comparative evaporation records are available for only a few places. As previously mentioned, records observed from floating pans are unreliable, and since the evaporation from a sunken land pan is almost identical with that from a floating pan of the same size, there is no purpose in using a floating pan.

Procedure

The location of the evaporation pan should be given careful consideration; otherwise, comparable and reliable data can not be obtained. The land pans should be installed in level areas unobstructed by trees and buildings, and if the observations are for the purpose of determining the equivalent reservoir evaporation, the location chosen should be representative of conditions at the reservoir. Isolated places, where no satisfactory water supply is available, or where it is not possible to secure the services of an intelligent observer, should be avoided. The equipment should be protected by a close mesh-wire fence. Fig. 5 is a diagram of a completely equipped Class *A* Weather Bureau Station. Although floating pans are not recommended, if it is necessary to install one for any reason, the location chosen should be in an area protected, to some extent, from the full force of the waves. The exposure to the wind, however, should represent the average conditions for the reservoir. The approximate elevation of the area chosen should be known, because evaporation varies with altitude.

The evaporation from the different types of pans is dependent, to some extent, on the color of the pan, and for this reason it is recommended that no paint, tar, or other coating, be used. To reduce the growth of algæ and other plants in the water, the pans should be cleaned at least once a month, and oftener if necessary. Cleaning the tanks also reduces the concentration of salts in the water, which otherwise would occur due to the evaporation.

In order to obtain comparable evaporation data, complete meteorological records should be taken and a standardized procedure should be followed in taking the observations. The meteorological record should include the air and water temperature, the humidity, the wind movement, the precipitation, and the evaporation loss. The following instruments are required:

Two sets of "maximum-and-minimum" thermometers for determining the mean air and water temperature; or
Two ordinary thermometers where observations can be taken at 12-hr intervals, or oftener;
One psychrometer for determining the humidity of the air;
One anemometer for recording the wind movement;
One rain gauge for measuring the precipitation;
One gauge for measuring the evaporation; and
One instrument shelter for the air thermometers and the psychrometer.

The standard Weather Bureau practice should be followed in taking the observations. Complete instructions for the operation of Pan (1) are given in *Circular L* of the Instrument Division of the U. S. Weather Bureau, and observations on the other types of pans should follow the same procedure. The mean water temperature and the relative humidity of the air are not observed by the Weather Bureau at its Class *A* stations, but these observations should be taken. The mean temperature of the water may be determined either by readings of "maximum-and-minimum" thermometers floating in the water with their bulbs immersed ¼ in. beneath the surface of the water, or by readings on an ordinary thermometer similarly exposed. The "maximum-and-minimum" thermometers require only a single reading daily to obtain a fair average of the temperature, whereas the ordinary thermometer must be read twice daily at 12-hr intervals for similar accuracy. Maximum thermometers are easily broken, however, and for this reason are not as satisfactory when the readings are taken by amateur observers. Where possible, the sling or rotating psychrometer should be used for determining the humidity, but where the readings can not be taken at regular intervals, a hair hygrometer may be used to obtain approximate results.

The velocity of the wind varies with the elevation above the ground surface. For this reason, a standard anemometer setting should be adopted for each type of evaporation pan in order to obtain comparable results. The standard practice at Class *A* Weather Bureau stations is to mount the anemometers on the grillage of timbers which supports the pan so that the cups of the anemometer are 6 in. above the top of the pan (see Fig. 3). At stations of the Bureau of Plant Industry, the anemometer is placed so that the cups are 24 in. above the top of the pan. Although no standard has been adopted

for installing the anemometers at stations where Pans (3) or (4) are used, it is suggested for uniformity that the anemometers be mounted so that their cups are 18 in. above the surface of the ground or the water. Either 3-cup or 4-cup anemometers, of the Weather Bureau pattern, may be used. The 3-cup anemometers are more accurate at high wind velocities, but near the ground the velocities are never so high that the 4-cup anemometer is not sufficiently accurate.

The rain gauge for evaporation stations should be of the Weather Bureau type and should be installed in accordance with the instructions in *Circular L* of the Weather Bureau. The evaporation gauges used in connection with the different pans are usually of special forms, but any gauge giving readings to within 0.001 ft should be satisfactory.

Observations should be taken at 7:00 A. M., according to the recommendations of the Weather Bureau, but where complete and accurate meteorological data require it, the readings should be taken oftener. Observations twice daily at 12-hr intervals (usually at 7:00 A. M. and 7:00 P. M.) give more accurate results, particularly as to the humidity. Where readings can not be taken at regular intervals, recording instruments should be used to obtain the records.

Conclusions

Records from floating pans are not as consistent or reliable as land-pan records; nor is the evaporation from a floating pan any nearer the evaporation from a large water surface than that from a sunken pan of the same size and shape.

Comparisons between the evaporation from Class *A* land pans of the Weather Bureau (Type (1)) and Colorado sunken pans (Type (3)) with the evaporation from large water surfaces, indicate that there is a definite relation between the pan and reservoir evaporation. For the Class *A* land pan, the factor for computing the reservoir evaporation from the pan evaporation is between 0.69 and 0.70, and for the Colorado pan, it is 0.78.

Comparison of the evaporation from different types of pans with that from large water surfaces of different sizes, shows that the size of the pan has a proportionately smaller effect on the evaporation as the size of the surface increases, and that when the diameter is greater than 12 ft, the size of the pan has practically no effect on the evaporation.

When all factors are considered, Pan (1) is probably better suited for evaporation investigations than the other evaporation pans. In order to obtain comparable evaporation results, standard equipment installed under representative conditions should be used, and a standard procedure should be followed in making the observations.

Acknowledgments

The author wishes to acknowledge the co-operation of those agencies and individuals who furnished evaporation data from their unpublished records, or who gave other information about the experiments.

APPENDIX

Summary of Data on Evaporation from Water Surfaces

For convenience of reference in this Appendix the various pans used in evaporation studies are identified by numbers as defined in the paper. For example, Pan (1) is the Class *A* land pan of the U. S. Weather Bureau, shown in Figs. 1 and 5; Pan (2) is the sunken pan of the U. S. Bureau of Plant Industry, shown in Fig. 6; Pan (3) is the Colorado sunken pan shown in Figs. 1, 3, and 7; Pan (4) is the floating pan of the U. S. Geological Survey shown in Fig. 8; and Pan (5) is simply Pan (1) used as a floating pan, as shown in Fig. 4.

East Park Reservoir, Stonyford, Calif.—The data in Table 1 are from records of the Bureau of Agricultural Engineering, United States Department of Agriculture, for observations made in July, 1930.[8] Water was maintained at a level approximately 2 in. below the rim in Pans (1) and (3). In Pans (2) and (4), the level was 3 in. below the rim.

TABLE 1.—Evaporation from Reservoir Compared with That from Pans; East Park Reservoir, Stonyford, Calif.

Item No.	Water surface	Dimensions: Area	Dimensions: Depth, in feet	Precipitation, in inches	Temperature, in Degrees, Fahrenheit: Air	Temperature, in Degrees, Fahrenheit: Water	Difference in vapor pressure, in inches	Velocity of wind, in miles per hour	Evaporation; total for the month, in inches	Ratio of evaporation, Item No. 1 to other items
(1)	(2)	(3)	(4) •	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	East Park Reservoir	1800 acres	....	0.0	82.12	80.72	0.6723	1.676	8.183	1.00
2	Pan (3)	3 ft. square	1.5	0.0	78.42	79.32	0.6395	1.940	9.498*	0.75
3	Pan (2)	Diameter, 4 ft.	3.0	0.0	78.42	79.77	0.6393	1.905	10.464	0.78
4	Pan (1)	Diameter, 4 ft.	0.83	0.0	78.42	73.83	0.4998	2.667	11.926	0.69
5	Pan (5)	Diameter, 4 ft.	0.83	0.0	78.53	77.91	0.5680	1.779	10.432	0.78

* Data for the period beginning July 19, 1930. The corresponding evaporation from the reservoir for the same period is 7.162.

Pans (2) and (3) were buried to within 3 in. and 1 in. of the top, respectively. Pan (1) was installed on a grillage of timbers, and Pan (5) was fixed in a raft. In Column (9), Table 1, all the anemometer readings have been reduced to surface velocities. East Park Reservoir is in a water-tight valley, geologically. Its elevation is 1 200 ft.

Comparison of Evaporation from Pan (1) and Various Other Types of Pans.—The data in Table 2 are largely self-explanatory. In Table 2(*a*) a comparison is afforded between evaporation from Pans (1) and (2) (see Columns (8) and (9)), a Class *A* land pan, and a sunken pan. This part of Table 2 was compiled from information presented[9] by Ivan E. Houk, M. Am. Soc. C. E., covering observations made during 1926, 1927, and 1928. Pan (1) was the standard type shown in Fig. 5, while Pan (2) was 4 ft in diameter

[8] *Technical Bulletin No. 271,* U. S. Dept. of Agriculture.
[9] *Transactions,* Am. Soc. C. E., Vol. 94 (1930), Table 9, p. 991.

TABLE 2.—COMPARISON BETWEEN EVAPORATION FROM PAN (1), AND VARIOUS OTHER TYPES OF PANS

Item No.	Month	Number of years of record	Precipitation, in inches	Temperature of air, in degrees Fahrenheit	Relative humidity (percentage)	Velocity of wind, in miles per hour	Evaporation: Total for Month, in Inches — Pan (1)		Ratio: Column (9) to Column (8)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	(a) Los Griegos, New Mexico, Column (9), Is Type (2); 4 Feet in Diameter and 2 Feet Deep								
	Pans						Type (1)	Type (2)	
1	Jan......	2	0.0	36.2	48	2.3	1.74	1.04	0.60
2	Feb......	2	0.33	41.1	50	3.6	3.06	2.12	0.69
3	March...	2	0.28	46.8	40	4.2	6.20	4.46	0.72
4	April.....	2	0.46	52.9	37	4.6	8.50	5.86	0.69
5	May.....	2	0.69	61.6	34	4.0	10.68	7.28	0.68
6	June.....	2	0.50	68.2	34	3.4	11.46	7.74	0.68
7	July.....	2	1.12	74.1	49	2.6	11.12	7.60	0.68
8	Aug.....	2	2.14	70.8	61	2.6	8.76	7.02	0.80
9	Sept.....	3	0.84	65.7	55	2.6	7.02	5.26	0.75
10	Oct.....	2	0.61	56.0	56	2.3	5.27	3.82	0.72
11	Nov.....	2	0.02	45.4	52	3.0	3.70	2.72	0.74
12	Dec.....	2	0.54	33.4	72	2.8	1.36	0.98	0.72
13	Totals...	......	7.53	....	....	....	78.87	55.90	....
14	Means...	......	....	54.3	49	3.2	....	....	0.71
	(b) Garnett, Colorado, Column (9) Is Type (2); 4 Feet in Diameter and 2 Feet Deep								
	Pans						Type (1)	Type (2)	
15	April.....	1	0.36	41.9*	....	4.2	....	....	0.79
16	May.....	2	1.14	48.4*	....	3.7	....	....	0.76
17	June.....	3	1.00	57.4*	....	3.4	....	....	0.78
18	July.....	3	2.16	61.6*	....	3.1	....	....	0.77
19	Aug.....	3	1.44	59.5*	....	1.8	....	....	0.76
20	Sept.....	3	1.00	53.5*	....	2.0	....	....	0.81
21	Oct.....	3	0.45	43.8*	....	1.9	....	....	0.80
22	Totals...	......	7.55	....	....	....	....	....	....
23	Means...	......	....	52.3*	....	2.9	....	....	0.78
	(c) Lincoln, Nebraska, Column (9) Is Briggs Pan, Type (2); 7.5 Feet in Diameter and 2 Feet Deep								
	Pans						Type (1)	Briggs Pan	
24	May.....	4	3.45	61	66	4.8	6.603	5.181†	0.78
25	June.....	4	4.38	72	65	3.6	8.789	6.386	0.73
26	July.....	4	1.73	78	60	3.0	10.250	7.094	0.69
27	Aug.....	4	3.14	74	65	2.7	8.592	5.921	0.69
28	Sept.....	4	2.61	66	66	3.5	6.612	4.612†	0.70
29	Oct.....	4	2.51	54	67	4.1	4.588	3.160	0.69
30	Totals...	......	17.82	....	....	....	45.434	32.354	....
31	Means...	......	....	68	65	3.6	....	....	0.71
	(d) Milford, Utah, Column (9) Is Type (2); 12 Feet in Diameter and 3 Feet Deep								
	Pans						Type (1)	Type (2)	
32	March...	1	0.90	41.3	54	4.4‡	3.98§	2.74§	0.69
33	April.....	2	0.64	49.3	53	4.3	8.42‖	5.54‖	0.66
34	May.....	2	0.82	57.7	39	4.9	11.19	7.74	0.69
35	June.....	2	0.02	65.2	38	4.8	14.71	9.92	0.67
36	July.....	2	0.66	73.3	39	4.2	14.02	9.40	0.67
37	Aug.....	3	0.23	70.0	45	4.6	13.39¶	9.02¶	0.67
38	Sept.....	3	0.21	60.0	41	4.9	10.75	7.23	0.67
39	Oct.....	3	1.12	50.1	44	3.7	6.59	4.40	0.67
40	Totals...	......	4.60	....	....	....	83.05	55.99	....
41	Means...	......	....	58.4	44	4.5	....	....	0.67

* Temperature of water. † Three-year record. ‡ From *Water Supply Paper 617*, U. S. Geological Survey, p. 36. § Twelve days, March, 1926, estimated. ‖ Twenty-three days, April, 1927, estimated. ¶ Four days, August, 1925, estimated.

TABLE 2.—(*Continued*)

Item No.	Month	Number of years of record	Precipitation, in inches	Temperature of air, in degrees Fahrenheit	Relative humidity (percentage)	Velocity of wind, in miles per hour	EVAPORATION: TOTAL FOR MONTH, IN INCHES		Ratio: Column (9) to Column (8)
							Pan (1)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
(*e*) NELSON RESERVOIR, MONTANA, COLUMN (9) IS TYPE (5); 4 FEET IN DIAMETER AND 10 INCHES DEEP									
	Pans						Type (1)	Type (5)	
42	May......	3	2.37	54.5	64	6.64	8.82	5.26	0.60
43	June......	3	5.06	65.9	64	5.80	9.57	6.37	0.67
44	July......	3	2.99	69.6	63	4.65	11.18	7.82	0.70
45	Aug......	3	0.95	64.2	59	4.69	11.36	8.67	0.76
46	Sept......	3	0.76	58.7	60	5.01	7.70	5.31	0.69
47	Oct......	3	0.09	47.9	66	5.32	4.26	2.78	0.65
48	Totals...	......	12.22	....	....	....	52.89	36.21	....
49	Means...	......	....	60.1	63	5.35	....	....	0.68
(*f*) OAKDALE, CALIFORNIA, COLUMN (9) IS TYPE (5); 4 FEET IN DIAMETER AND 10 INCHES DEEP									
	Pans						Type (1)	Type (5)	
50	June......	1	0.00**	70.8	....	5.9	13.373	8.233	0.62
51	July......	1	0.00	78.5	....	5.8	17.034	10.849	0.64
52	Aug......	1	0.00	73.6	....	5.5	14.651	9.303	0.63
53	Sept......	1	0.06	68.8	....	4.6	10.351	6.839	0.66
54	Totals...	......	0.06	....	....	....	55.409	35.224	....
55	Means...	......	....	72.9	....	5.4	....	....	0.64
(*g*) FALL RIVER MILLS, CALIFORNIA, COLUMN (9) IS TYPE (5); 4 FEET IN DIAMETER AND 8 INCHES DEEP									
	Pans						Type (1)	Type (5)	
56	Jan......	5	2.30	33.1	....	1.5	....	1.085	....
57	Feb......	5	2.79	38.6	....	2.0	2.034††	1.755	0.86
58	March...	5	2.23	44.4	....	2.5	3.635‡‡	3.199	0.88
59	April......	5	1.73	48.2	....	3.1	4.903	4.255	0.87
60	May......	5	1.13	55.4	....	2.9	7.670	6.356	0.83
61	June......	5	0.59	63.4	....	2.8	10.209	7.768	0.76
62	July......	5	0.00**	69.3	....	2.8	13.032	9.226	0.71
63	Aug......	5	0.06	66.1	....	2.8	11.445	7.732	0.68
64	Sept......	5	0.57	57.5	....	2.4	7.235	5.349	0.74
65	Oct......	5	0.85	50.8	....	1.8	4.581	3.316	0.72
66	Nov......	5	3.39	42.1	....	1.6	....	1.659	....
67	Dec......	5	1.72	34.1	....	1.5	....	1.368	....
68	Totals...	......	17.36	....	....	....	64.744	48.956	....
69	Means...	......	....	50.2	....	2.3	....	....	0.76

** Trace. †† Two-year record. ‡‡ Four-year record.

and 2 ft deep, with 3 in. of the tank above ground. Water in Pan (2) was maintained at ground level. The anemometer cups were fixed 24 in. above ground, and readings were taken generally at 5:00 P. M. Los Griegos, N. Mex., is at an elevation of 4 970 ft.

Table 2(*b*) is from unpublished data in the office of the State Engineer of Colorado, made available through the courtesy of R. J. Tipton, Assoc. M. Am. Soc. C. E. Observations were made in 1927, 1928, and 1930. Pans (1) and (2) were the same dimensions as given in Table 2(*a*), with the difference that Pan (2) was buried to within 2 in. above ground. Mariotte control apparatus[10] was used to maintain the level in the pans. A 3-cup anemometer was fixed at 18 in. above the ground. Garnett, Colo., is at Elevation 7 700.

[10] For description, see *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 962.

Table 2(*c*)[11] is from data observed during the four years, 1917 to 1920, inclusive. The Briggs pan (Type (2)) was 7.5 ft in diameter and 2.0 ft deep, and was set with the rim flush with the ground. Water was maintained 4 in. below the rim in both pans, and the anemometer cups were 5 in. above Pan (1). The relative humidity readings in Column (6) were taken at Lincoln, Nebr. (3 miles from the pans) at 7:00 A. M. and 7:00 P. M. The pans, in this case, were at Elevation 1 150.

Table 2(*d*)[12] is from data observed during the three years, 1925, 1926, and 1927. Pan (2) was 12 ft in diameter, was set with the rim 3 in. above ground, and the water was maintained at ground level. The relative humidity in Column (6), Table 2(*d*), was taken from the record of Modena, Utah. Evaporation readings were taken at 8:00 A. M. The elevation of Milford, Utah, is 4 962.

Table 2(*e*) shows[13] some data and observations made during the three years, 1921, 1922, and 1923. Water in the floating pan, Type (5), was 6 in. deep, and the surface was maintained at the same elevation as the water in the reservoir. This pan was 350 ft from shore; the land pan, Type (5), was on a small knoll, 100 yd from shore. Anemometer readings were begun in 1922, with standard setting. Precipitation records (Column (4)) are those of Malta, Mont., and the relative humidity (Column (6)) readings are those at Havre, Mont. Nelson Reservoir is at Elevation 2 215.

The data pertaining to the floating pans in Tables 2(*f*) and 2(*g*) are from unpublished records furnished by the courtesy of the U. S. Weather Bureau. The land-pan records are from Climatological Data, California Section, U. S. Weather Bureau. The data in Table 2(*f*) were recorded during 1921. The land pan (Type (1)) was placed on a knoll and was filled with water to about 7 in., the anemometer cups being 6 in. above the rim. Observations were taken daily at 7:00 A. M. The floating pan (Type (5)) was exposed on a raft and filled to a depth of 6 to 7 in. of water. Oakdale, Calif., is at Elevation 215.

Data in Table 2(*g*) were recorded during the five years, 1925 to 1930. The land pan (Type (1)) was filled with water to a depth of about 7 in., anemometer cups being 11 in. above the rim. The floating pan (Type (5)) was submerged 4 in. and was filled with water to a depth of 6 in. Observations were taken daily at 7:30 A. M. Fall River Mills, Calif., is at Elevation 3 340, and the ground surrounding the station is flat.

Comparison Between Evaporation from Different Sizes of Pans.—Data compiled in Table 3(*a*) are from records of the U. S. Bureau of Agricultural Engineering, covering the years 1926, 1927, and 1928.[8] The wind velocity given is at the surface of the ground or water in each case. The wind velocities for Pans (3) and (4) are the same as for the reservoir. The elevation of Fort Collins, Colo., is 5 000 ft.

[11] Comp. from *Monthly Weather Review*, Vol. 48, December, 1920, p. 715.

[12] Comp. from Climatological Data, Utah Section, U. S. Weather Bureau.

[13] Comp. from Climatological Data, Montana Section, and from the Annual Rept. of the Chief of the U. S. Weather Bureau; see *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 275.

TABLE 3.—Comparison Between Evaporation from Different Sizes of Pans

Item No.	Month	Number of years of record	Pan A: Temperature, in degrees Fahrenheit: Air	Pan A: Temperature, in degrees Fahrenheit: Water	Pan A: Difference in vapor pressure, in inches	Pan A: Wind velocity, in miles per hour	Pan A: Total evaporation for the month, in inches	Pan B: Temperature of water, in degrees Fahrenheit	Pan B: Difference in vapor pressure, in inches	Pan B: Total evaporation for the month, in inches	Pan B: Ratio: Column (8) to Column (11)	Pan C: Temperature of water, in degrees Fahrenheit	Pan C: Difference in vapor pressure, in inches	Pan C: Total evaporation for the month, in inches	Pan C: Ratio: Column (8) to Column (15)	Pan D: Temperature of water, in degrees Fahrenheit	Pan D: Difference in vapor pressure, in inches	Pan D: Wind velocity, in miles per hour	Pan D: Total evaporation for the month, in inches	Pan D: Ratio: Column (8) to Column (20)	Precipitation, in inches
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
(a) Fort Collins, Colorado																					
1	Pans.......		Reservoir: 85 Feet in Diameter and 6.75 Feet Deep					Pan (4): 3 Feet Square and 18 Inches Deep				Pan (3): 3 Feet Square and 18 Inches Deep				Pan (1): 4 Feet in Diameter and 10 Inches Deep					
2	April.......	2	45.6	50.8	0.1691	2.67	3.14	50.1	0.1677	4.08	0.77	49.6	0.1846	4.18	0.75	48.4	0.1857	3.71	5.21	0.60	1.408
3	May.......	2	58.3	62.5	0.2526	2.11	4.40	62.1	0.2530	5.80	0.76	61.6	0.2746	5.78	0.76	60.4	0.2752	2.91	6.94	0.63	2.176
4	June.......	2	62.3	68.5	0.2812	1.16	3.75	67.9	0.2800	4.94	0.76	66.6	0.2848	4.85	0.77	65.2	0.2699	1.70	5.42	0.69	2.565
5	July.......	2	70.5	75.8	0.3776	1.04	4.99	75.0	0.3709	6.59	0.76	74.1	0.3956	6.54	0.76	73.0	0.3842	1.58	7.21	0.69	1.229
6	Aug.......	2	67.2	72.3	0.3372	1.03	4.47	71.7	0.3313	5.82	0.77	70.3	0.3450	5.96	0.75	68.7	0.3203	1.56	6.26	0.71	1.128
7	Sept.......	3	57.4	64.7	0.3063	1.17	4.19	63.9	0.3014	5.22	0.80	60.9	0.2891	4.90	0.86	58.6	0.2607	1.76	5.10	0.82	0.522
8	Oct.......	3	48.8	54.4	0.1998	1.34	2.71	53.7	0.1974	3.59	0.76	51.4	0.1938	3.30	0.82	50.1	0.1876	2.05	3.77	0.72	0.792
9	Nov.......	3	37.9	41.1	0.0974	1.68	1.42	40.0	0.0898	1.74	0.82	37.1	0.0747	1.44	0.99	37.2	0.0782	2.60	1.84	0.77	0.474
10	Totals.....	......	......	......	......	......	29.07	......	......	37.78	......	......	......	36.95	......	......	......	......	41.75	......	10.291
11	Means.....	....	56.0	61.3	0.2526	1.52	......	60.6	0.2489	......	0.77	59.0	0.2553	......	0.79	57.7	0.2452	2.23	......	0.70	......
(b) Salt Creek Bridge, California (Circular, Floating, Type (5) Pan)																					
12	Pans.......		Pan (5): 12 Feet in Diameter and 10 Inches Deep					Pan (5): 6 Feet in Diameter and 10 Inches Deep				Pan (5): 4 Feet in Diameter and 10 Inches Deep				Pan (5): 2 Feet in Diameter and 10 Inches Deep					
13	May.......	5*	88.2	82.8	0.646	4.2	12.12	82.4	0.631	12.80	0.95	82.6	0.640	14.79	0.82	82.9	0.648	......	14.54	0.83	......
14	June.......	30*	86.0	82.6	0.628	5.0	10.89	82.0	0.604	11.61	0.94	82.0	0.609	13.29	0.82	81.9	0.601	......	13.83	0.79	......
15	July.......	6*	87.8	84.1	0.646	4.5	11.90	83.3	0.618	12.37	0.96	83.5	0.622	14.26	0.84	83.3	0.616	......	14.54	0.82	......
16	Totals.....	......	......	......	......	......	34.91	......	......	36.78	......	......	......	42.34	......	......	......	......	42.91	......	......
17	Means.....	......	86.53	82.84	0.633	4.83	......	82.24	0.609	......	0.95	82.29	0.615	......	0.82	82.23	0.609	......	......	0.81	......

* Days.

(c) SALTON SEA, CALIFORNIA (CIRCULAR LAND PAN, TYPE (1))

18	Pans		Pan (1): 6 Feet in Diameter and 10 Inches Deep									Pan (1): 4 Feet in Diameter and 10 Inches Deep				Pan (1): 2 Feet in Diameter and 10 Inches Deep					
19	April	18*	75.9	75.5	0.589	4.93	11.58	...	...	...	...	76.1	0.616	11.52	1.01	76.1	0.610	...	13.44	0.86	...
20	May	31*	79.9	79.2	0.572	7.08	13.02	...	...	...	...	79.5	0.587	13.95	0.93	79.3	0.576	...	16.86	0.77	...
21	June	30*	83.8	83.7	0.710	7.03	13.62	...	...	...	...	83.3	0.694	15.48	0.88	82.9	0.681	...	18.81	0.72	...
22	July	30*	88.5	88.9	0.690	6.67	13.70	...	...	...	...	88.9	0.675	15.81	0.87	88.9	0.661	...	19.62	0.70	...
23	Totals	...	...	...	...	...	51.92	...	...	...	...	...	...	56.76	...	...	...	...	68.73	...	...
24	Means	...	82.68	82.50	0.6453	6.72	...	...	...	...	...	82.57	0.6455	...	0.92	82.40	0.6339	...	...	0.76	...

(d) BRAWLEY, CALIFORNIA (CIRCULAR LAND PAN, TYPE (1))

25	Pans		Pan (1): 6 Feet in Diameter and 10 Inches Deep									Pan (1): 2 Feet in Diameter and 10 Inches Deep									
26	March 5†	4*	70.7	70.2	0.504	0.9	0.193‡	...	...	...	...	...	...	...	...	71.6	0.539	...	0.283‡	0.68	...
27	March 11†	4*	68.4	67.3	0.476	1.0	0.228‡	...	...	...	...	...	...	...	...	67.6	0.488	...	0.295‡	0.77	...
28	March 17†	3*	71.2	66.2	0.374	2.1	0.240‡	...	...	...	...	...	...	...	...	71.2	0.496	...	0.315‡	0.76	...
29	March 26†	5*	64.4	65.1	0.362	4.5	0.283‡	...	...	...	...	...	...	...	...	66.0	0.382	...	0.346‡	0.82	...
30	April 2†	5*	66.0	65.1	0.453	2.4	0.264‡	...	...	...	...	...	...	...	...	65.5	0.461	...	0.362‡	0.73	...
31	April 16‡	5*	61.9	65.5	0.390	3.1	0.276‡	...	...	...	...	...	...	...	...	66.6	0.413	...	0.354‡	0.78	...
32	April 23‡	5*	79.9	75.7	0.701	1.7	0.350‡	...	...	...	...	...	...	...	...	77.4	0.748	...	0.453‡	0.77	...
33	April 30‡	5*	80.8	75.9	0.654	3.0	0.409‡	...	...	...	...	...	...	...	...	77.2	0.693	...	0.532‡	0.77	...
34	Means	...	70.4	68.9	0.489	2.3	0.280‡	...	...	...	...	...	...	...	...	70.4	0.528	...	0.368‡	0.76	...

(e) MECCA, CALIFORNIA (CIRCULAR LAND PAN, TYPE (1))

35	Pans		Pan (1): 6 Feet in Diameter and 10 Inches Deep									Pan (1): 2 Feet in Diameter and 10 Inches Deep									
36	March 5‡	5	68.4	65.8	0.417	2.4	0.272‡	...	...	...	...	...	...	...	...	66.4	0.429	...	0.362‡	0.75	...
37	March 12‡	5	67.5	64.0	0.413	2.3	0.244‡	...	...	...	...	...	...	...	...	64.4	0.421	...	0.335‡	0.73	...
38	March 17‡	3	69.8	68.7	0.425	3.3	0.264‡	...	...	...	...	...	...	...	...	69.4	0.441	...	0.398‡	0.66	...
39	March 25‡	4	64.4	65.3	0.362	3.7	0.276‡	...	...	...	...	...	...	...	...	64.8	0.350	...	0.378‡	0.73	...
40	April 2‡	5	66.4	65.8	0.468	2.9	0.311‡	...	...	...	...	...	...	...	...	64.9	0.449	...	0.449‡	0.69	...
41	April 9‡	3	74.8	71.2	0.465	4.0	0.346‡	...	...	...	...	...	...	...	...	72.3	0.492	...	0.496‡	0.70	...
42	April 16‡	5	67.1	66.9	0.394	3.1	0.283‡	...	...	...	...	...	...	...	...	66.9	0.394	...	0.382‡	0.74	...
43	April 23‡	5	82.6	73.2	0.669	3.2	0.449‡	...	...	...	...	...	...	...	...	73.8	0.685	...	0.642‡	0.70	...
44	April 29‡	4	81.7	75.6	0.622	2.7	0.417‡	...	...	...	...	...	...	...	...	76.3	0.646	...	0.559‡	0.75	...
45	Means	...	71.4	68.5	0.471	3.1	0.318	...	...	...	...	...	...	...	...	68.8	0.479	...	0.445‡	0.71	...

* Days. † Period ending. ‡ Per 24 hours.

Tables 3(*b*) and 3(*c*) were compiled from unpublished data for 1910 furnished by the courtesy of the U. S. Weather Bureau. The pans in Table 3(*b*) were floated on a raft in an arm of the Salton Sea (approximate Elevation − 210), and were kept well immersed. The anemometer cups were approximately 8 in. higher than the rims of the pans.

The land pans in Table 3(*c*) were set on a platform 8 or 10 ft above the Salton Sea (approximate Elevation − 200), about ½ mile from the shore. Water was maintained in them at a depth of approximately 7 in. The anemometer cups were approximately 8 in. higher than the rims of the pans.

TABLE 4.—COMPARISON BETWEEN EVAPORATION FROM A 12-FOOT CIRCULAR SUNKEN PAN (TYPE (2)) AND VARIOUS OTHER TYPES

Item No.	Type of pan	Diameter, in feet	Depth, in feet	Depth of water in pan, in feet	Depth of setting, in feet	PERIOD OF OBSERVATION				EVAPORATION FOR THE PERIOD, IN INCHES		Ratio: Col. (11) to Col. (12)
						From	To	Except	Days of record	Item No. 1	Given pan	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	2	12.0	3.0	2.75	2.75	11/15/15	9/30/17	....	684	85.47	85.47	1.00
2	2	1.0	3.0	2.75	2.75	11/15/15	9/30/17	12/ 4/16 to 5/ 7/17	428	79.68	126.59	0.63
3	2	2.0	3.0	2.75	2.75	11/15/15	11/13/16	....	363	49.16	63.12	0.78
4	2	3.39	3.0	2.75	2.75	11/15/15	9/17/17	11/21/16 to 3/11/17	183	80.51	96.74	0.83
5	2	6.0	3.0	2.75	2.75	11/15/15	9/30/17	....	684	85.47	92.87	0.92
6	2	9.0	3.0	2.75	2.75	11/15/15	11/13/16	....	363	49.16	49.63	0.99
7	3	2.0†	3.0	2.75	2.75	4/17/16	9/24/17	11/13/16 to 5/29/17	328	68.18	89.49	0.76
8	3	3.0†	3.0	2.75	2.75	11/15/15	9/24/17	11/13/16 to 5/ 8/17	502	78.33	98.63	0.79
9	1	4.0	0.83	0.62	0.00‡	11/15/15	9/30/17	12/13/15 to 3/ 6/16 and 11/13/16 to 4/11/17	452	81.69	116.09	0.70
10	1	2.0	0.83	0.62	0.00‡	4/11/17	9/30/17	....	172	32.53	54.91	0.59
11	1	6.0	0.83	0.62	0.00‡	4/11/17	7/30/17	....	110	20.05	26.66	0.75
12	2	2.0	0.5	0.25	0.25	6/ 5/16	7/30/17	10/ 9/16 to 5/28/17	189	43.47	55.62	0.78
13	2	2.0	1.0	0.75	0.75	6/ 5/16	9/24/17	10/ 9/16 to 5/28/17	244	55.24	70.16	0.79
14	2	2.0	1.5	1.25	1.25	6/ 5/16	9/24/17	10/ 9/16 to 5/28/17	215	55.24	70.69	0.78
15	2	2.0	2.0	1.75	1.75	6/ 5/16	7/30/17	10/ 9/16 to 5/28/17	189	43.47	55.30	0.79
16	2	2.0	6.0	5.75	5.75	6/ 5/16	9/24/17	10/ 9/16 to 5/28/17	215	55.24	70.39	0.78
17	2	6.0	1.0	0.75	0.75	6/ 5/16	10/ 9/16	....	97	28.49	30.48	0.93
18	2	6.0	2.0	1.75	1.75	6/ 5/16	10/ 9/16	....	97	28.49	30.36	0.94
19	4	3.0†	1.5	1.25	1.25§	4/17/16	9/17/17	*	....	64.06	72.04	0.89
20	5	0.83	0.83	0.75	0.75	7/31/17	9/17/17	....	72	10.27	13.42	0.77
21	5	2.0	1.0	0.75	0.75	5/10/17	9/17/17	*	....	14.29	16.04	0.89
22	5	4.0	1.0	0.75	0.75	7/31/17	9/17/17	....	48	10.27	10.63	0.97
23	5	6.0	1.0	0.75	0.75	5/10/17	8/27/17	*	....	6.57	6.39	1.03

* Intermittent periods. † Square. ‡ Above ground. § Depth not given, but assumed to be standard.

Tables 3(*d*) and 3(*e*) are from unpublished records of 1910 furnished by the courtesy of the U. S. Weather Bureau. Water in the pans was maintained at an approximate depth of 8 in. In Table 3(*d*) anemometer cups were approximately 7 in., and in Table 3(*e*), 8.8 in., higher than the rim. The water surface was fully exposed. Brawley, Calif., is at Elevation − 110, and Mecca, Calif., is at Elevation − 189.

Comparison Between Evaporation from 12-Foot Circular Sunken Pan (Type (2)) and Various Other Types.—All the data in Table 4 were observed at Denver, Colo. (Elevation 5 346), during the years 1915, 1916, and 1917.[14]

[14] Comp. from *Journal of Agricultural Research,* Vol. X, No. 5, July 30, 1917, and from *Transactions,* Am. Soc. C. E., Vol. 90 (June, 1927), Table 15, p. 308.

General Comparisons: Land Pans and Floating Pans.—Table 5(*a*) has been compiled from three sources: *Bulletin 9* of the Department of Engineering, State of California; Climatological Data, California Section, U. S. Weather Bureau; and, *Transactions,* Am. Soc. C. E., Vol. 90 (June, 1927). Buena Vista Lake has an area of about 10 000 acres and is at Elevation 420. The observations were made in 1920, the temperature and precipitation records being taken at Bakersfield, Calif. The land pan—Type (3)—was sunk to within 3 in. of the top; the floating pan—Type (4)—was always submerged to within 6 in., of the top, or less. The depth of water was maintained at 3 in. below the top of both pans.

Table 5(*b*) is compiled from records in the *Water Supply Papers* of the U. S. Geological Survey, collected between 1916 and 1925. The Class *A* land pan (Type (1)) and the floating pan of the U. S. Geological Survey (Type (4)) were both of standard dimensions, as shown in Figs. 5 and 8, respectively. The floating pan was set in a lake 20 by 150 ft and from 8 to 10 ft lower than the land pan. Observations were taken at 7:00 A. M. Austin, Tex., is at Elevation 475.

Tables 5(*c*), 5(*d*), and 5(*e*) are from the set-up of identical pans at Morena Reservoir, Barrett Reservoir, and Lower Otay Reservoir, California, during 1928. The precipitation for all three was taken at the Barrett Dam and the temperature at Escondido, Calif. Some of the records were furnished by the courtesy of H. N. Savage, M. Am. Soc. C. E., and others were taken from Climatological Data, California Section, U. S. Weather Bureau. The elevations of the stations are as follows: Morena, 3 000 ft; Barrett, 1 600 ft; anod Lower Otay, 500 ft, above sea level.

Table 5(*f*) was compiled from data presented[15] by Charles H. Lee, M. Am. Soc. C. E., and from the Annual Reports of the Chief of the Weather Bureau. The floating pan (Type (4)), 3 ft square and 10 in. deep, was placed on a raft in Owens River. Water was kept approximately 2 in. below the rim. The deep land pan (Type (2)) was set with the top flush with the ground. In this pan water was never more than 4 in. below the rim. The shallow land pan (Type (3)) was set with the top 5 in. above ground and the water level was maintained approximately 2 in. below the rim. The anemometer cups were 42 ft above ground. Some of the evaporation records were partly estimated. Independence, Calif., is at Elevation 3 907; the period of record was from 1908 to 1911.

Table 5(*g*) was compiled from published and unpublished data recorded[16] by Mr. C. M. O'Neil, of the Canadian Pacific Railroad Company, during 1923 and 1924. The dimensions of land and floating pans were alike. The land pans were sunk in the ground to within 2 in. of their tops. Water was kept at ground level. In the floating pans, water was kept 4 in. below the rim or slightly above the level of submergence. The anemometer cups were 2 ft above ground. Temperature and wind velocity data were taken from plotted data. Brooks Reservoir, Alberta, Canada (Elevation 2 450), has an area of about 40 acres.

---

[15] *Transactions,* Am. Soc. C. E., Vol. LXXVIII (1915), pp. 177–178.

[16] *Loc. cit.,* Vol. 90 (June, 1927), pp. 343–351.

TABLE 5.—GENERAL COMPARISON OF EVAPORATION DATA FROM LAND PANS AND FLOATING PANS

Item No.	Month	Number of years of record	Precipitation, in Inches		Temperature, in Degrees Fahrenheit: Air	Water			Total Evaporation for the Month, in Inches			Ratio of Evaporation: Column (11) to Column (10)	Column (12) to Column (11)	Column (12) to Column (10)	Relative humidity (percentages)	Wind velocity, in miles per hour
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
	(a) Bakersfield, California (Buena Vista Lake); Pans (3) and (4), 3 Feet Square and 2 Feet Deep															
1	Pan.........								Pan (3)	Pan (4)	Lake					
2	Jan.........	1	0.51		47.6	......	......	......	1.82	1.82	......	1.00	......	......	......	......
3	Feb.........	1	1.58		52.6	......	......	......	2.45	2.50†	......	1.02	......	......	......	......
4	March.......	1	2.28		53.4	......	......	......	2.83	2.89†	......	1.02	......	......	......	......
5	April.......	1	0.14		59.0	......	......	......	5.99	6.78	......	1.13	......	......	......	......
6	May.........	1	0.00		68.3	......	......	......	9.05	10.58	......	1.17	......	......	......	......
7	June........	1	0.44		74.5	......	......	......	10.00	11.72	......	1.17	......	......	......	......
8	July........	1	0.00		79.4	......	......	......	10.14	12.61	7.3	1.24	0.58	0.72	......	......
9	Aug.........	1	0.00*		81.2	......	......	......	9.62	11.27	7.0	1.17	0.62	0.73	......	......
10	Sept........	1	0.00		71.7	......	......	......	7.21	8.46	7.1	1.17	0.84	0.98	......	......
11	Oct.........	1	0.63		60.4	......	......	......	4.88	5.42	4.6	1.11	0.85	0.94	......	......
12	Nov.........	1	0.16		55.6	......	......	......	2.29	2.44	2.0	1.06	0.82	0.87	......	......
13	Dec.........	1	0.76		49.0	......	......	......	1.56	1.50	......	0.96	......	......	......	......
14	Totals......	......	6.50		......	......	......	......	67.84	77.99	28.0	......	......	......	......	......
15	Means.......	......	....		62.7	......	......	......	......	......	......	1.15	0.70	0.82	......	......
	(b) Austin, Texas (Pans (1) and (4), Standard Dimensions)															
16	Pans........					Pan (1)	Pan (4)		Pan (1)	Pan (4)						
17	Jan.........	6	2.50		48.0	45.9	47.8	......	2.51	1.92	......	0.77	......	......	87.0	3.2
18	Feb.........	8	2.61		52.0	47.9	51.6	......	3.05	2.38	......	0.79	......	......	85.0	3.6
19	March.......	8	2.10		58.8	53.5	57.8	......	5.33	3.88	......	0.74	......	......	78.4	3.5
20	April.......	9	4.41		66.2	60.6	65.4	......	6.03	4.92	......	0.83	......	......	81.8	2.5
21	May.........	9	4.47		73.4	69.0	72.6	......	6.91	5.46	......	0.79	......	......	83.3	1.7
22	June........	9	3.51		80.9	75.0	78.7	......	7.86	6.00	......	0.76	......	......	79.9	1.2
23	July........	9	2.74		83.2	76.8	80.2	......	8.77	7.07	......	0.81	......	......	78.7	1.3
24	Aug.........	9	1.06		83.6	77.2	80.3	......	8.62	6.88	......	0.80	......	......	79.0	1.2
25	Sept........	8	4.22		77.6	72.0	75.7	......	6.39	5.01	......	0.79	......	......	81.3	1.3
26	Oct.........	8	3.82		68.2	61.6	66.5	......	5.12	4.03‡	......	0.79	......	......	82.0	1.4
27	Nov.........	8	2.29		57.2	52.0	56.4	......	3.03	2.31	......	0.76	......	......	83.6	1.5
28	Dec.........	8	1.79		51.0	47.9	50.4	......	2.54	2.01	......	0.79	......	......	83.9	2.8
29	Totals......	......	35.52		......	......	......	......	66.16	51.87	......	......	......	......	......	......
30	Means.......	......	....		66.7	61	65.3	......	......	......	......	0.78	......	......	82.0	2.1

* Trace. † Values estimated. ‡ Seven years of record.

* Trace. † Values estimated. ‡ Seven years of record.

(c) MORENA RESERVOIR, CALIFORNIA (PANS (3) AND (4), 3 FEET SQUARE AND 1.5 FEET DEEP

31	Pans........							Pan (3)	Pan (4)						
32	Jan.........	1	0.80	54.6	......	......	......	3.65	3.07	......	0.84	......	......	......	......
33	Feb.........	1	1.73	53.2	......	......	......	3.07	2.27	......	0.74	......	......	......	......
34	March.......	1	0.73	56.9	......	......	......	3.96	3.25	......	0.82	......	......	......	......
35	April.......	1	0.17	57.2	......	......	......	6.72	6.37	......	0.95	......	......	......	......
36	May.........	1	0.31	64.3	......	......	......	8.86	7.48	......	0.84	......	......	......	......
37	June........	1	0.00	66.3	......	......	......	9.44	8.60	......	0.91	......	......	......	......
38	July........	1	0.00	69.9	......	......	......	11.14	10.03	......	0.90	......	......	......	......
39	Aug.........	1	0.00	70.0	......	......	......	11.23	9.93	......	0.88	......	......	......	......
40	Sept........	1	0.00	69.5	......	......	......	9.71	8.82	......	0.91	......	......	......	......
41	Oct.........	1	0.32	61.8	......	......	......	7.00	5.97	......	0.85	......	......	......	......
42	Nov.........	1	1.09	56.8	......	......	......	4.23	3.65	......	0.86	......	......	......	......
43	Dec.........	1	2.90	53.0	......	......	......	2.76	......	......	......	......	......	......	......
44	Totals......	......	8.05	......	......	......	......	81.77	69.44	......	......	......	......	......	......
45	Means.......	......	....	61.1	......	......	......	......	......	......	0.88	......	......	......	......

(d) BARRETT RESERVOIR, CALIFORNIA (PANS (3) AND (4), 3 FEET SQUARE AND 1.5 FEET DEEP

46	Pans........							Pan (3)	Pan (4)						
47	Jan.........	1	0.80	54.6	......	......	......	2.32	2.36	......	1.02	......	......	......	......
48	Feb.........	1	1.73	53.2	......	......	......	2.67	2.49	......	0.93	......	......	......	......
49	March.......	1	0.73	56.9	......	......	......	3.61	3.56	......	0.99	......	......	......	......
50	April.......	1	0.17	57.2	......	......	......	6.06	6.06	......	1.00	......	......	......	......
51	May.........	1	0.31	64.3	......	......	......	7.53	§	......	§	......	......	......	......
52	June........	1	0.00	66.3	......	......	......	9.49	§	......	§	......	......	......	......
53	July........	1	0.00	69.9	......	......	......	10.65	§	......	§	......	......	......	......
54	Aug.........	1	0.00	70.0	......	......	......	10.30	§	......	§	......	......	......	......
55	Sept........	1	0.00	69.5	......	......	......	9.49	§	......	§	......	......	......	......
56	Oct.........	1	0.32	61.8	......	......	......	5.75	5.30	......	0.92	......	......	......	......
57	Nov.........	1	1.09	56.8	......	......	......	3.79	3.16	......	0.83	......	......	......	......
58	Dec.........	1	2.90	53.0	......	......	......	2.41	2.01	......	0.83	......	......	......	......
59	Totals......	......	8.05	......	......	......	......	74.07	24.94	......	......	......	......	......	......
60	Means.......	......	....	61.1	......	......	......	......	......	......	0.94	......	......	......	......

§ Records lost because pans were out of order.

TABLE 5.—(*Continued*)

Item No.	Month	Number of years of record	Precipitation, in Inches		Temperature, in Degrees Fahrenheit: Air	Water			Total Evaporation for the Month, in Inches			Ratio of Evaporation: Column (11) to Column (10)	Column (12) to Column (11)	Column (12) to Column (10)	Relative humidity (percentages)	Wind velocity, in miles per hour
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
	(*e*) Lower Otay Reservoir, California (Pans (3) and (4), 3 Feet Square and 1.5 Feet Deep)															
61	Pans								Pan (3)	Pan (4)						
62	Jan.	1	0.80		54.6	......	......	......	2.09	2.72	......	1.30	......	......	......	......
63	Feb.	1	1.73		53.2	......	......	......	2.45	2.63	......	1.07	......	......	......	......
64	March	1	0.73		56.9	......	......	......	3.92	3.74	......	0.95	......	......	......	......
65	April	1	0.17		57.2	......	......	......	5.93	5.52	......	0.93	......	......	......	......
66	May	1	0.31		64.3	......	......	......	5.66	5.48	......	0.97	......	......	......	......
67	June	1	0.00		66.3	......	......	......	7.80	7.13	......	0.91	......	......	......	......
68	July	1	0.00		69.9	......	......	......	9.98	8.46	......	0.85	......	......	......	......
69	Aug.	1	0.00		70.0	......	......	......	8.87	8.29	......	0.93	......	......	......	......
70	Sept.	1	0.00		69.5	......	......	......	7.00	6.64	......	0.95	......	......	......	......
71	Oct.	1	0.32		61.8	......	......	......	4.99	5.21	......	1.04	......	......	......	......
72	Nov.	1	1.09		56.8	......	......	......	3.65	3.65	......	1.00	......	......	......	......
73	Dec.	1	2.90		53.0	......	......	......	2.18	2.18	......	1.00	......	......	......	......
74	Totals	......	8.05		......	......	......	......	64.52	61.65	......	......	......	......	......	......
75	Means	......	....		61.1	......	......	......	......	......	......	0.96	......	......	......	......
	(*f*) Independence, California (Standard, Type (4), Floating Pan, and Type (2) Land Pan, 3½ Feet in Diameter and 4 Feet Deep; Type (3) Land Pan, 3 Feet Square and 10 Inches Deep)															
76	Pans								Pan (2)	Pan (3)	Pan (4)					
77	Jan.	3	2.14		39.9	......	......	......	2.15‖	2.25¶	1.67	......	0.74	0.78	70	6.5
78	Feb.	3	1.19		39.5	......	......	......	2.72‖	2.25¶	2.42	......	1.08	0.89	64	7.5
79	March	3	0.35		49.1	......	......	......	4.78	6.06‖	4.52	......	0.75	0.95	49	9.2
80	April	3	0.14		56.5	......	......	......	6.66	8.81‖	6.87	......	0.78	1.03	38	8.7
81	May	3	0.01		62.4	......	......	......	7.70‖	10.43‖	8.63	......	0.83	1.12	33	8.1
82	June	2	0.04		72.2	......	......	......	8.20	11.95¶	10.00	......	0.84	1.22	29	7.5
83	July	2	0.12		77.4	......	......	......	8.10	12.55¶	9.45	......	0.75	1.17	30	6.6
84	Aug.	3	0.08		75.7	......	......	......	8.50‖	11.25‖	7.70	......	0.68	0.91	29	6.0
85	Sept.	3	0.47		67.2	......	......	......	7.25‖	8.65‖	6.07	......	0.70	0.84	33	6.8
86	Oct.	3	0.13		57.1	......	......	......	5.08‖	5.70‖	3.87	......	0.68	0.76	41	6.0
87	Nov.	3	0.10		46.7	......	......	......	3.20‖	3.32‖	2.49	......	0.75	0.78	53	5.7
88	Dec.	3	1.43		34.6	......	......	......	2.18‖	1.60‖	1.53	......	0.96	0.70	75	7.0
89	Totals	......	6.21		......	......	......	......	66.52	84.82	65.22	......	......	......	......	......
90	Means	......	....		56.5	......	......	......	......	......	......	......	0.77	0.98	45	7.1

‖ Two-year record. ¶ One-year record.

(g) BROOKS RESERVOIR AND LAKE NEWELL, ALBERTA, CANADA (PAN (2) AND PAN (5), 4 FEET IN DIAMETER AND 20 INCHES DEEP)

91	Pans		Brooks	Newell		Brooks Reservoir Pan (2)	Brooks Reservoir Pan (5)		Lake Newell Pan (2)	Brooks Reservoir Pan (2)	Brooks Reservoir Pan (5)		Brooks	Newell**		
92	May	2	0.83	0.98	......	......	......	......	5.53††	5.74††	......	......	......	0.96	......	3.2
93	June	2	2.93	2.74	59	61††	62††	......	5.65††	5.04††	5.86††	......	1.16	1.12	......	2.6
94	July	2	1.67	1.62	66	72††	69††	......	6.72	6.50	7.97	......	1.23	1.03	......	2.4
95	Aug.	2	2.28	2.18	60	65††	64††	......	4.37	4.61	6.19	......	1.34	0.95	......	2.0
96	Sept.	2	0.55	0.36	54	54††	54††	......	4.08	3.50	4.90	......	1.40	1.16	......	1.6
97	Totals	......	8.26	7.88	......	......	......	......	26.35	25.39	24.92	......	......	......	......	......
98	Means	......	......	......	59.8	63.0	62.2	......	......	......	......	......	1.27	1.14	......	2.4

(h) CHESTNUT HILL RESERVOIR, BOSTON, MASS. (THREE TYPE (5) PANS)

99	Pans:					Pan (5)	Pan (5)	Pan (5)	Pan (5)	Pan (5)	Pan (5)					
100	Diameter, in feet					1	1	10	1	1	10					
101	Depth, in feet					2	8	10	2	8	10					
102	June	18‡‡	......	......	......	72.5	72.9	73.3	5.29	5.13	4.58	......	0.89	0.87	......	......
103	July	31‡‡	......	......	......	77.2	77.7	77.2	7.99	8.04	7.14	......	0.89	0.89	......	......
104	Aug.	31‡‡	......	......	......	74.4	75.0	74.4	6.93	7.05	7.41	......	1.05	1.07	......	......
105	Sept.	30‡‡	......	......	......	65.7	66.6	66.0	5.64	5.81	5.13	......	0.88	0.91	......	......
106	Oct.	28‡‡	......	......	......	57.0	57.7	56.6	3.39	3.47	2.79	......	0.80	0.82	......	......
107	Totals	......	......	......	......	......	......	......	29.24	29.50	27.05	......	......	......	......	......
108	Means	......	......	......	......	69.4	70.0	69.5	......	......	......	......	0.92	0.93	......	......

(i) CHESTNUT HILL RESERVOIR, BOSTON, MASS. (THREE TYPE (5) PANS)

109	Pans:					Pan (5)	Pan (5)	Pan (5)	Pan(5)§§	Pan(5)§§	Pan(5)§§					
110	Diameter, in feet					1	1	10	1	1	10					
111	Depth, in feet					2	8	10	2	8	10					
112	June 23–24	24‖‖	......	......	64.5	69.4	70.0	69.0	0.490	0.500	0.420	......	0.84	0.86	50.0	11.9
113	June 24–25	24‖‖	......	......	70.2	70.0	70.2	70.2	0.320	0.320	0.270	......	0.84	0.84	60.5	7.2
114	July 8–9	24‖‖	......	......	77.8	79.6	80.2	79.6	0.320	0.320	0.242	......	0.76	0.76	74.4	6.2
115	July 9–10	24‖‖	......	......	74.4	77.7	78.2	77.8	0.310	0.320	0.255	......	0.80	0.82	76.2	8.0
116	July 10–11	24‖‖	......	......	70.6	77.1	77.8	77.3	0.300	0.310	0.280	......	0.90	0.93	73.4	6.8
117	Aug. 7–8	24‖‖	......	......	69.3	74.8	75.2	74.8	0.260	0.260	0.285	......	1.10	1.10	77.4	4.0
118	Sept. 11–12	24‖‖	......	......	59.1	66.3	67.0	65.7	0.205	0.210	0.160	......	0.76	0.78	76.9	2.7
119	Oct. 14–15	24‖‖	......	......	53.5	55.6	56.0	55.5	0.130	0.140	0.100	......	0.71	0.77	83.8	6.9
120	Totals	......	......	......	......	......	......	......	2.335	2.380	2.012	......	......	......	......	......
121	Means	......	......	......	67.4	71.3	71.8	71.2	......	......	......	......	0.85	0.86	71.6	6.7

** Pan (5) records for Lake Newell are not listed. †† Records are for 1924 only. ‡‡ Days. ‖‖ Hours. §§ Evaporation, in inches per 24 hours.

TABLE 6.—SUMMARY OF THE COMPARISON OF THE EVAPORATION FROM DIFFERENT TYPES OF PANS

Item No.	Type of pan	Location	Diameter, in feet	Depth of pan, in feet	Depth of water, in feet	Depth of setting, in feet	Elevation, in feet above sea level	Number of seasons of record	Ratio of Evaporation from a Given Pan or Reservoir to Evaporation from: Class A land pan, Type (1)	Sunken land pan, Type (2)	Colorado sunken land pan, Type (3)	Sunken circular land pan, Type (2)	U. S. Geological Survey floating pan, Type (4)	U. S. Weather Bureau floating pan Type (5)	Observer
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1	(2)	Denver, Colo.	1.0	3.0	2.75	2.75	5 346	3	1.12	1.49	1.26	......	......	......	Sleight
2	(2)	Denver, Colo.	2.0	3.0	2.75	2.75	5 346	2	0.90	1.21	1.02	......	......	......	"
3	(2)	Denver, Colo.	3.39	3.0	2.75	2.75	5 346	3	0.85	1.13	0.96	......	......	......	"
4	(2)	Denver, Colo.	6.0	3.0	2.75	2.75	5 346	3	0.77	1.02	0.87	......	......	......	"
5	(2)	Denver, Colo.	9.0	3.0	2.75	2.75	5 346	2	0.71	0.95	0.80	......	......	......	"
6	(2)	Denver, Colo.	12.0	3.0	2.75	2.75	5 346	3	0.70	0.94	0.80	......	......	......	"
7	(2)	Denver, Colo.	2.0	0.5	0.25	0.25	5 346	2	0.90	1.20	1.02	......	......	......	"
8	(2)	Denver, Colo.	2.0	1.0	0.75	0.75	5 346	2	0.89	1.19	1.01	......	......	......	"
9	(2)	Denver, Colo.	2.0	1.5	1.25	1.25	5 346	2	0.90	1.20	1.02	......	......	......	"
10	(2)	Denver, Colo.	2.0	2.0	1.75	1.75	5 346	2	0.90	1.20	1.02	......	......	......	"
11	(2)	Denver, Colo.	2.0	6.0	5.75	5.75	5 346	2	0.90	1.19	1.01	......	......	......	"
12	(2)	Denver, Colo.	6.0	1.0	0.75	0.75	5 346	1	0.75	1.00	0.85	......	......	......	"
13	(2)	Denver, Colo.	6.0	2.0	1.75	1.75	5 346	1	0.75	1.00	0.85	......	......	......	"
14	(3)	Denver, Colo.	2.0*	3.0	2.75	2.75	5 346	2	0.92	1.23	1.04	......	......	......	"
15	(3)	Denver, Colo.	3.0*	3.0	2.75	2.75	5 346	3	0.88	1.18	1.00	......	......	......	"
16	(1)	Denver, Colo.	2.0	0.83	0.62	0.00§	5 346	1	1.19	1.58	1.34	......	......	......	"
17	(1)	Denver, Colo.	4.0	0.83	0.62	0.00§	5 346	3	1.00	1.33	1.13	......	......	......	"
18	(1)	Denver, Colo.	6.0	0.83	0.62	0.00§	5 346	1	0.94	1.25	1.06	......	......	......	"
19	(5)	Salton Sea, Calif.	2.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	1.02	Bigelow
20	(5)	Salton Sea, Calif.	4.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	1.00	"
21	(5)	Salton Sea, Calif.	6.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	0.87	"
22	(5)	Salton Sea, Calif.	12.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	0.82	"
23	(1)	Salton Sea, Calif.	2.0	0.83	0.6	0.00§	—200	4¶	1.21	......	......	......	......	......	"
24	(1)	Salton Sea, Calif.	4.0	0.83	0.6	0.00§	—200	4¶	1.00	......	......	......	......	......	"
25	(1)	Salton Sea, Calif.	6.0	0.83	0.6	0.00§	—200	4¶	0.92	......	......	......	......	......	"
26	(4)	Independence, Calif.	3.0*	0.83	0.67	0.5	3 907	3	......	......	0.97**	......	......	......	Lee
27	(4)	Santa Clara Valley, Calif.	3.0*	1.0	0.83	......	......	2	......	......	0.71††	......	......	......	Duryea
28	(5)	Brooks Reservoir, Alberta, Canada	4.0	1.67	1.33	1.33	2 450	2	......	......	......	1.27	......	......	O'Neil
29	(5)	Brooks Reservoir, Alberta, Canada	4.0	1.67	1.33	1.33	2 450	2	......	......	......	1.14‡‡	......	......	"
30	(‡)	Newell Reservoir, Alberta, Canada	16.430†	......	......	......	2 485	7	......	......	......	0.95	0.70	......	"

* Square. † Acres. ‡ Lake or Reservoir. § Exposed. ‖ Well immersed. ¶ Months. ** Sunken land pan, Column (12), 10 in. deep. †† Sunken land pan, Column (12), 12 in. deep; floating pan on raft in reservoir; ratio, average of six land pans to three floating pans. ‡‡ At Lake Newell, Alberta, Canada.

* Square. † Acres. ‡ Lake or Reservoir. § Exposed. ‖ Well immersed. ¶ Months. ** Sunken land pan, Column (12), 10 in. deep. †† Sunken land pan, Column (12), 12 in. deep; floating pan on raft in reservoir; ratio, average of six land pans to three floating pans. ‡‡ At Lake Newell, Alberta, Canada.

31	(‡)	Buena Vista Lake, Calif.	10 000†	......	......	......	420	1	......	......	0.82	......	0.81	......	Harding
32	(‡)	Lake Elsinore, Calif.	5 500†	......	......	......	1 261	1	......	......	......	......	......	......	"
33	(‡)	Lake Hodges, Calif.	1 317†	......	......	......	315	0.5	......	......	0.96	......	1.08	......	Lippincott
34	(‡)	Cuyamaca Lake, Calif.	978†	......	......	......	4 600	2¶	......	......	......	......	......	......	"
35	(‡)	Cuyamaca Lake, Calif.	978†	......	......	......	4 600	2¶	......	......	1.19	......	......	......	"
36	(2)	Stonyford, Calif.	4.0	3.0	2.75	2.75	1 200	1¶	0.88	......	0.96	1.00	......	1.01	Rohwer
37	(2)	Los Griegos, N. Mex.	4.0	2.0	1.75	1.75	4 970	2	0.71	......	......	......	......	......	Houk
38	(2)	Garnett, Colo.	4.0	2.0	......	1.83	7 700	3	0.78	......	......	......	......	......	Tipton
39	...	Means, Items No. 36, 37, and 38	......	......	......	......	......	......	0.79	......	0.96	1.00	......	1.01	....
40	(2)	Denver, Colo.	6.0	1.0	0.75	0.75	5 346	3	0.75	1.00	0.85	......	......	......	Sleight
41	(2)	Denver, Colo.	6.0	2.0	1.75	1.75	5 346	1	0.75	1.00	0.85	......	......	......	"
42	(2)	Denver, Colo.	6.0	3.0	2.75	2.75	5 346	1	0.77	1.02	0.87	......	......	......	"
43	...	Means, Items Nos. 40, 41, and 42	......	......	......	......	......	......	0.76	1.01	0.86	......	......	......	....
44	(2)	Lincoln, Nebr.	7.5	2.0	1.67	2.0	1 150	4	0.71	......	......	......	......	......	Loveland
45	(2)	Denver, Colo.	9.0	3.0	2.75	2.75	5 346	2	0.71	0.95	0.80	......	......	......	Sleight
46	(2)	Denver, Colo.	12.0	3.0	2.75	2.75	5 346	3	0.70	0.94	0.80	......	......	......	"
47	(2)	Milford Utah.	12.0	3.0	......	2.75	4 962	3	0.67	......	......	......	......	......	White
48	(2)	Fort Collins, Colo.	85.0	6.75	6.4	6.4	5 000	3	0.70	......	0.79	......	0.77	......	Rohwer
49	(‡)	East Park Reservoir, Stonyford, Calif.	1 800†	......	......	......	1 200	1¶	0.69	......	0.75	0.78	......	0.78	"
50	...	Means, Items Nos. 44 to 49	......	......	......	......	......	......	0.70	0.94	0.78	0.78	0.77	0.78	....
51	(3)	Stonyford, Calif.	3.0*	1.5	1.3	1.4	1 200	1¶	0.91	......	1.00	1.04	......	1.04	Rohwer
52	(3)	Fort Collins, Colo.	3.0*	1.5	1.33	1.33	5 000	3	0.89	......	1.00	......	0.98	......	"
53	(3)	Denver, Colo.	3.0*	3.0	2.75	2.75	5 346	3	0.88	1.18	1.00	......	......	......	Sleight
54	...	Means, Items Nos. 51, 52, and 53	......	......	......	......	......	......	0.89	1.18	1.00	1.04	0.98	1.04	....
55	(1)	Stonyford, Calif.	4.0	0.83	0.6	0.00§	1 200	1¶	1.00	......	1.10	1.14	......	1.14	Rohwer
56	(1)	Fort Collins, Colo.	4.0	0.83	0.62	0.00§	5 000	3	1.00	......	1.13	......	1.11	......	"
57	(1)	Denver, Colo.	4.0	0.83	0.62	0.00§	5 346	3	1.00	1.33	1.13	......	......	......	Sleight
58	...	Means, Nos. 55, 56, and 57	......	......	......	......	......	......	1.00	1.33	1.12	1.14	1.11	1.14	....

* Square. † Acres. § Exposed. ¶ Months.

TABLE 6.—(*Continued*)

Item No.	Type of pan	Location	Diameter, in feet	Depth of pan, in feet	Depth of water, in feet	Depth of setting, in feet	Elevation, in feet above sea level	Number of seasons of record	Ratio of Evaporation from a Given Pan or Reservoir to Evaporation from: Class *A* land pan, Type (1)	Sunken land pan, Type (2)	Colorado sunken land pan, Type (3)	Sunken circular land pan, Type (2)	U. S. Geological Survey floating pan, Type (4)	U. S. Weather Bureau floating pan, Type (5)	Observer
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
59	(4)	Buena Vista, Calif.......	3.0*	2.0	1.75	1.5	420	1	......	......	1.15	......	1.00	......	Harding
60	(4)	Austin, Tex.............	3.0*	1.5	......	......	475	9	0.78	......	......	......	1.00	......	Geological Survey.
61	(4)	Lower Otay Reservoir, Calif..............	3.0*	1.5	1.25	......	500	1	......	......	0.96	......	1.00	......	Savage
62	(4)	Barrett Lake, Calif......	3.0*	1.5	1.25	......	1 600	1	......	......	0.94	......	1.00	......	"
63	(4)	Morena Lake, Calif......	3.0*	1.5	1.25	......	3 000	1	......	......	0.88	......	1.00	......	"
63	(4)	Fort Collins, Colo.......	3.0*	1.5	1.25	1.25	5 000	3	0.91	......	1.02	......	1.00	......	Rohwer
65	(4)	Denver, Colo...........	3.0*	1.5	1.25	1.25§§	5 346	2	......	......	......	......	1.00	1.09	Sleight
66	...	Means, Items Nos. 59 to 65..................	......	......	......	......	......	......	0.84	......	0.99	......	1.00	1.09	....
67	(5)	Stonyford, Calif.........	4.0	0.83	0.5	......	1 200	1¶	0.88	......	0.96	1.00	......	1.00	Rohwer
68	(5)	Nelson Reservoir, Mont..	4.0	0.83	0.5	0.5	2 215	4	0.68	......	......	......	......	1.00	Reclamation Bureau.
69	(5)	Fall River Mills, Calif....	4.0	0.67	0.5	0.33	3 340	5	0.76	......	......	......	......	1.00	Pac. Gas & Elec. Co.
70	(5)	Denver, Colo...........	4.0	1.0	0.75	0.75	5 346	1	......	......	......	......	0.92	1.00	Sleight
71	...	Means, Items Nos. 67 to 70..................	......	......	......	......	......	......	0.77	......	0.96	1.00	0.92	1.00	....
72	(5)	Salton Sea, Calif........	2.0	0.83	0.6	‖	—210	3¶	......	......	......	......	......	1.01	Bigelow
73	(5)	Denver, Colo...........	2.0	1.0	0.75	0.75	5 346	1	......	......	......	......	1.00	1.08	Sleight
74	...	Means, Items Nos. 72 and 73..................	......	......	......	......	......	......	......	......	......	......	1.00	1.04	....
75	(5)	Salton Sea, Calif........	6.0	0.83	0.6	‖	—210	......	......	......	......	......	......	0.87	Bigelow
76	(5)	Denver, Colo...........	6.0	1.0	0.75	0.75	5 346	......	......	......	......	......	0.86	0.94	Sleight
77	...	Means, Items Nos. 75 and 76..................	......	......	......	......	......	......	......	......	......	......	0.86	0.90	....

* Square. ‖ Well immersed. §§ Assumed to be standard depth. ¶ Months.

Tables 5(h) and 5(i) are from observations made[17] in 1885, on floating pans of the same type, but of different diameters and depths. All pans were floated in Chestnut Hill Reservoir, Boston, Mass. (Elevation 129), supported by a raft 20 ft wide and 40 ft long. They were filled with water to within 3 in. of their tops and submerged to within 6 in. of their tops (as scaled from photographs). Readings were taken daily for Table 5(h) and hourly for Table 5(i). The anemometer was 30.5 ft above the water surface.

Summary.—Table 6 is a summary tabulation of conclusions drawn from the preceding tables.

---

[17] *Transactions*, Am. Soc. C. E., Vol. XV (1886), p. 581.

EVAPORATION FROM RESERVOIR SURFACES

By Robert Follansbee,[18] M. Am. Soc. C. E.

Synopsis

This paper contains the results of all available evaporation records, not only in the United States and outlying possessions, but also in foreign countries, reduced to reservoir surface evaporation by means of coefficients which are stated for each record. These results are given in summarized form, together with records of temperature, wind velocity, and relative humidity, as far as available. A total of 210 evaporation records are presented.

After the summary the relative effect of temperature, wind velocity, and relative humidity is shown by comparison between pairs of records in which two factors are the same and the third is widely different.

A brief discussion of the variation in evaporation throughout the United States concludes the paper.

Introduction

For many years records of evaporation have been kept by different organizations, which have used pans varying not only in size and shape, but in setting. Few, if any, of these records give directly the evaporation from reservoir surfaces, but must be corrected by coefficients that vary with the size of the pan and its setting. By reducing the records to reservoir surface evaporation, the data in which engineers are chiefly interested will be obtained, and a common basis for comparison of evaporation under varying climatic conditions will be available. The writer has compiled, not only for the United States and its possessions, but also for foreign countries, all evaporation records of which he has knowledge, and with them the available related data on temperature, relative humidity, and wind velocity.

Coefficients to Reduce Observed Evaporation to Reservoir Surface

To reduce the observed evaporation to reservoir surface evaporation, use has been made of Mr. Rohwer's paper, published as a part of this Symposium, which brings together the results of many experiments on the subject.

Of the many evaporation records available, the greater number have been taken by pans of the following types:

(1) The United States Weather Bureau Class *A* pan, which is a circular pan, 4 ft in diameter and 10 in. deep, placed on an open timber platform resting on the ground.

Note.—This paper is published with the approval of the Director of the U. S. Geological Survey.

[18] Dist. Engr., U. S. Geological Survey, Denver Colo.

(2) The United States Bureau of Plant Industry pan, which is a circular pan, 6 ft in diameter and 2 ft deep, sunk in the ground with its rim 3 in. above the surface.

(3) The floating pan, which may be either 3 ft square and 18 in. deep, or a circular pan, 4 ft in diameter and 10 in. deep, suspended from an enclosing raft with some form of wooden shield to protect the water in the pan from wave action.

United States Weather Bureau Class A Pan.—In addition to the records cited by Mr. Rohwer, those for the Salton Sea, in California, have been used for an approximate comparison. The work done at Salton Sea during 1909 and 1910 does not lend itself to an exact comparison between evaporation from large water surfaces and evaporation from land pans of the Class *A* type, but the following rough approximation can be made.

A Class *A* pan was placed on a platform 2 ft above the water surface at a point in Salton Sea 7 500 ft from shore. This setting may be considered to approximate that of a Class *A* pan on a wooden platform on the ground. One year's record from this pan showed an evaporation of 106.45 in. The actual evaporation from Salton Sea during that period has been computed variously as 69 to 75 in. With a mean value of 72 in., the coefficient for the Class *A* pan is $72 \div 106.45 = 0.68$.

Another Class *A* pan was placed on a platform 2 ft above the water surface, at a point in Salton Sea 500 ft from shore. This showed an evaporation of 108.65 in. For this pan the coefficient was $72 \div 108.65 = 0.66$. The mean relative humidity was 60 per cent.

Table 7 is a summary of Mr. Rohwer's results as well as those at Salton Sea; and of the work of the late R. B. Sleight, Assoc. M. Am. Soc. C. E.,

TABLE 7.—COEFFICIENTS TO REDUCE CLASS *A* PAN RECORDS TO RESERVOIR SURFACE EVAPORATION

Location	Years of record	Comparison with	Coefficient	Temperature, in degrees Fahrenheit	Wind velocity, in miles per hour	Relative humidity (percentages)
Denver, Colo.	1915–17	12-ft ground pan	0.70	....	....	64
Fort Collins, Colo.	1926–28	85-ft reservoir	0.70	47	1.6	68
East Park Reservoir, California	1930	1 800-acre reservoir	0.68	80	1.7	47
Milford, Utah	1925–27	12-ft ground pan	0.70	50	4.3	41
Lincoln, Nebr.	1917–20	7½-ft ground pan	0.67	51	4.1	65
Salton Sea, California	1909–10	Indirect method	0.68	....	....	60
Salton Sea, California	1909–10	Indirect method	0.66	....	....	60

at Denver, Colo. In view of the results indicated, the coefficient to reduce the U. S. Weather Bureau Class *A* pan records to reservoir surface evaporation has been taken by the writer as 0.69.

It should be noted that the experiments were made under climatic conditions having a range of relative humidity from 41% to 68%, a range in temperature from 47° to 80° F, and a range in wind velocity from 1.6 to 4.3 miles per hour. As an increase of more than one-half in relative humidity

and similar ranges in temperature and wind velocities showed little change in the coefficient to be applied to Class *A* pans, it appears that for an increase in relative humidity to 70% or 80%, such as is found in the eastern part of the United States, the coefficient will be substantially the same.

United States Bureau of Plant Industry Pans.—The only direct comparison between a Bureau of Plant Industry pan and a large water surface was that made by Sleight in the Denver experiments, in which he found the coefficient to be 0.94.[19]

Floating Pans.—Theoretically, records from a floating pan of galvanized iron should give closely and consistently the evaporation from the water surface in which it floats. Practically, however, it has been found difficult to maintain accurate records, owing to the action of waves, which causes water to enter the pan or to spill out of it. Furthermore, a constant factor tending to increase the observed evaporation is the wetted strip around the inside of the pan just above the normal water surface, produced by wave action within the pan itself. To show the inconsistency of the results of experiments on floating pans, it is only necessary to reduce the records presented by Mr. Rohwer (to which have been added a few compiled by the writer) to the common basis of reservoir evaporation.

Direct comparisons between (1) floating pans, 3 ft square and 18 in. deep, and (2) floating circular pans, 4 ft in diameter and 10 in. deep, with large water surfaces, show the following results:

Location	Square Pan	Circular Pan
Denver, Colo.	0.88	....
Fort Collins, Colo.	0.77	....
East Park Reservoir, California	....	0.78
Lake Elsinore, California	0.84	....
Buena Vista Lake, California	0.70	....
Upper Otay Reservoir, California	0.87	....

Indirect comparisons by means of records from Class *A* pans give the following results:

Location	Square Pan	Circular Pan
Oakdale, Calif.	....	0.92
Fall River Mills, Calif.	....	0.52
Austin, Tex.	0.89	....
Granite Reef, Ariz.	....	0.81

These inconsistent results are practically the same for both types of floating pans, and an average of all coefficients (except that for Fall River Mills), gives 0.83 as the factor to reduce records from floating pans of the two types considered to evaporation from large water surfaces.

Egyptian Records.—The Egyptian Government has maintained evaporation records at points in the Nile Valley from Alexandria to the Southern Sudan. These records have been taken both by floating pans and Piche evaporimeters used in a meteorological screen. To determine the reduction factors, comparisons between Piche tubes and both floating and land pans

[19] "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 308.

were made. As it was not found practicable to experiment with pans larger than 2 m square, that size was used as the best available approximation to an open water surface. At Helwan Observatory, on the hills overlooking the Nile Valley, about 15 miles from Cairo, the factor to reduce the Piche records to a 2-m land pan was found to be 0.66. At Abbassiya, near Cairo, the factor to reduce Piche records to a 2-m floating pan square was 0.50, and, for a 1-m floating pan, the factor was 0.88.

By considering the relation between floating and land pans as given by Sleight[20] in his first report on the Denver experiments and combining these with the Helwan observations, the factor to reduce Piche records to those from a 1-m floating-pan record was 0.59. At Abbassiya, the corresponding factor was 0.57. The engineers of the Egyptian Government, therefore, used a factor of 0.50 for the Piche tubes; for their 1-m floating-pan records they used 0.88. In so doing, it was stated that the evaporation would not be underestimated. The factors were determined for each month of the year and there was no evidence of any seasonal variation, the probable departure of the monthly ratios from the mean of the year being 3% for the pans and 10% for the Piche tubes.[21]

Turkestan Records.—Lyman E. Bishop, M. Am. Soc. C. E., who spent two years in Turkestan, furnished evaporation records taken by the Soviet Government since 1920 at experiment stations located at Fergana, Andidjan, Khodjent, and Osh, in the Fergana Valley, in Southern Turkestan. This is a region with an irrigated area of 1 600 000 acres and an additional area of 750 000 acres of seeped land. Although no actual records are available, the temperatures are stated to increase gradually from about 5° F, in January, to 120° to 130° F, in July and August, with a monthly mean for these months of about 95 degrees.

The annual rainfall is from 10 to 12 in., all of which occurs between November and March. It is a region of little wind, and as the valley is almost enclosed by high mountains, the evaporation from the irrigated land on which a large quantity of water is used, and from the seeped land, causes a humid atmosphere, particularly at the evaporation pans.

The evaporation records were taken by pans about 6.5 ft square and 3 ft deep set in the ground. By comparison with Sleight's experiments at Denver, the coefficient to reduce the records to reservoir surface evaporation has been taken as 0.95.

Reduction of Pan Records to Reservoir Surface Evaporation

The foregoing coefficients (as well as some others determined from Sleight's experiments), which have been used by the writer, are based on mean values covering several months, and are shown by the experiments themselves to vary from month to month. For some types of pans, furthermore, the mean coefficient itself may be considerably in error, as shown by the inconsistent results. It is with these facts in mind that the observed

[20] *Journal of Agricultural Research*, Vol. X, No. 5, July 30, 1917.

[21] "The Nile Basin," Vol. 1, Cairo, Egypt, Ministry of Public Works, 1931.

TABLE 8.—Summary of Evaporation Records Reduced to Reservoir Surface Evaporation

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity: Near-by U. S. Weather Bureau Station: Height, in feet	Wind Velocity: Near-by U. S. Weather Bureau Station: Velocity, in miles per hour	Wind Velocity: At the pan, in miles per hour	Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches: April–September	Reservoir Surface Evaporation, in Inches: Annual	Reservoir Surface Evaporation, in Inches: Maximum per month	Percentage range of annual evaporation	Coefficient for pan
(a) **Atlantic Seaboard**												
Gardiner, Me.	100	1915–24	45	.....	.....	2.2	..	18.10	24.26	4.56	92–111	0.69
Voorheesville, N. Y.	330	1917–30	48	115	7.5	1.9	74	19.66	25.98	4.71	92–107	0.69
Ithaca, N. Y.	800	1918–30	47	100	9.1	1.8	78	17.11	22.54	4.17	90–114	0.69
Runyon, N. J.	50	1924–30	52	183	11.0	2.5	71	19.78	27.85	4.88	92–106	0.69
Pleasantville, N. J.	40	1924–30	51	172	16.5	2.9	74	23.02	31.81	5.40	84–104	0.69
Washington, D. C.	280	1915–17	54	85	6.5	2.3	69	23.52	34.53	4.87	97–103	0.69
Chapel Hill, N. C.	500	1921–30	61	.....	.....	1.1	69	20.03	28.56	4.71	93–109	0.69
Birmingham, Ala.	650	1910	63	.....	7.0	....	72	32.18	42.99	6.25	......	0.83
(b) **Great Lakes**												
Grand River Lock, Wis.	780	1905–12	46	.....	.....	.....	..	21.64	28.57	5.75	85–105	0.83
Menasha, Wis.	740	1906–12	47	.....	.....	....	..	12.29	17.54	3.88	73–115	0.83
Madison, Wis.	860	1906–11	46	78	10.0	....	75	12.91	19.82	3.04	97–102	0.83
Wooster, Ohio	1 000	1916–29	49	.....	.....	2.3	..	18.25	24.85	5.21	85–112	0.69
(c) **Mississippi Valley**												
Centreville, Minn.	880	1919–27	45	.....	.....	4.0	..	24.11	30.97	7.37	97–107	0.69
Iowa City, Iowa	610	1907–10	46	.....	.....	.....	..	19.59	30.08	5.36	93–112	0.83
Columbus, Ohio	763	1918–30	52	230	10.7	2.0	74	19.21	26.81	4.94	89–108	0.69
Cincinnati, Ohio	520	1910	54	51	6.8	....	68	29.46	38.19	6.02	......	0.83
Clarksburg, W. Va.	1 030	1923–30	53	.....	.....	2.6	79	20.65	26.60	6.05	94–117	0.69
Columbia, Mo., No. 1	750	1916–27	54	84	8.0	1.5	71	20.28	28.13	5.31	88–114	0.69
Columbia, Mc., No. 2	750	1916–26	54	84	8.0	2.9	71	26.31	35.82	6.85	......	0.69
(d) **Gulf Coast**												
Silverhill, Ala.	250	1918–30	67	161	7.3	1.8	78	25.35	38.27	5.73	90–111	0.69
Crowley, La.	21	1910–19	68	.....	.....	2.8	77	30.76	46.90	7.18	96–106	0.94
Beeville, Tex.	214	1922–30	70	.....	.....	5.7	..	37.64	56.57	10.64	91–114	0.95
(e) **North Dakota**												
University, Grand Forks	820	1905–20	38	58	9.8	....	79	21.72	27.07	5.82	90–117	0.83
Williston	1 875	1909–16	39	48	8.3	5.5	74	32.12	38.79	8.45	87–109	0.94
Dickinson	2 543	1907–20	40	.....	.....	6.3	..	31.19	39.21	9.38	85–130	{0.94 0.95
Mandan	1 750	1914–20	40	57	10.0	5.6	72	32.08	39.80	8.62	93–112	0.94
Edgeley	1 468	1907–20	40	56	13.9	7.0	77	27.92	35.90	7.84	89–114	{0.94 0.95
Hettinger	2 253	1911–20	41	.....	.....	6.2	75	31.04	39.60	10.48	82–127	0.94
(f) **South Dakota**												
Newell	2 880	1908–29	45	.....	.....	6.6	..	33.63	44.63	10.08	83–123	0.94
Rapid City	3 240	1916–21	46	.....	.....	2.0	58	25.61	36.43	7.02	92–112	0.69
Ardmore	3 557	1913–20	44	.....	.....	6.4	62	35.40	46.59	9.11	83–113	0.94
(g) **Nebraska**												
Lincoln	1 250	{1895–1910 1917–20	52	81	10.7	4.1	70	33.89	43.53	11.66	76–113	0.95
Lincoln	1 250	1917–30	51	81	10.0	4.1	69	32.06	42.04	9.92	87–118	0.69
North Platte A	3 000	1907–30	48	51	9.2	7.5	68	38.58	46.31	11.84	85–123	{0.94 0.95
North Platte B	2 820	1917–30	48	51	9.2	5.7	69	35.48	43.11	8.86	90–108	0.94
Mitchell (Lake Minatare)	4 120	1918–21	47	6	8.4	....	..	27.13	33.60	6.75	......	0.69
Mitchell (Sunflower)	4 070	1909–17	47	6	7.9	....	68	27.55	34.95	7.75	......	0.69
Mitchell	4 080	1911–29	47	.....	.....	7.0	..	34.17	41.78	9.28	83–118	0.94

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity: Near-by U. S. Weather Bureau Station: Height, in feet	Wind Velocity: Near-by U. S. Weather Bureau Station: Velocity, in miles per hour	Wind Velocity: At the pan, in miles per hour	Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches: April-September	Reservoir Surface Evaporation, in Inches: Annual	Reservoir Surface Evaporation, in Inches: Maximum per month	Percentage range of annual evaporation	Coefficient for pan
(*h*) Kansas												
Lawrence	825	1916–19	55	.....	.....	4.4	..	31.25	44.80	7.62	......	0.69
Manhattan	1 010	1924–29	54	.....	.....	3.4	..	30.14	42.65	8.06	91–112	0.69
Wichita	1 300	1918–27	56	158	13.0	4.2	68	34.60	49.62	9.27	92–113	0.69
Hays	2 000	1907–29	53	.....	.....	8.0	65	43.67	61.97	12.25	81–122	0.94 / 0.95
Colby	3 135	1914–30	51	.....	.....	6.3	64	38.86	54.43	10.77	83–114	0.94
Garden City	2 830	1908–30	54	.....	.....	8.0	60	49.97	68.27	13.08	83–111	0.94 / 0.95
Tribune	3 543	1918–30	51	.....	.....	6.1	..	42.57	57.35	11.53	88–107	0.69
(*i*) Oklahoma												
Lake Lawtonka	....	1913–17	60	.....	.....	....	..	37.27	49.45	9.82	......	0.83
Lawton	1 111	1916–20	61	.....	.....	6.4	..	41.75	54.35	11.42	87–113	0.94
Woodward	1 900	1914–20	57	.....	.....	8.2	..	45.73	58.53	10.84	89–108	0.94
(*j*) Texas												
Austin	475	1916–29	68	148	7.5	2.0	68	28.72	42.47	8.49	77–116	0.83
Austin	475	1916–30	68	148	7.5	2.0	68	30.77	45.83	8.50	82–125	0.69
San Antonio	700	1907–12	70	300	6.8	....	68	43.46	63.71	10.14	93–109	0.95
Dilley	586	1928–30	71	.....	.....	2.6	65	39.63	57.02	9.16	......	0.69
Laredo	455	1917–18	73	.....	.....	3.8	64	44.44	67.47	9.39	......	0.69
Dalhart	4 000	1908–20	54	.....	.....	7.9	61	50.67	67.78	12.04	89–108	0.94 / 0.95
Spur	....	1922–30	61	.....	.....	6.0	61	42.84	62.44	10.80	81.114	0.95
Chillicothe	1 406	1912–19	63	.....	.....	7.2	60	46.91	68.57	12.20	86–114	0.94
Amarillo	3 680	1907–19	56	49	13.0	9.6	56	49.00	66.00	12.63	84–109	0.94
Big Springs	2 396	1915–20	63	.....	.....	6.7	59	55.01	75.65	12.56	91–109	0.94
Balmorhea	....	1930–31	64	.....	.....	4.4	..	40.31	59.07	7.55	......	0.95
El Paso	3 700	1889–93	64	175	8.3	....	36	49.95	71.16	10.79	......	0.83
(*k*) Montana												
Malta (Nelson Reservoir)	2 215	1917–23	42	.....	.....	5.7	..	30.67	36.85	.....	......	0.83
Malta (Nelson Reservoir)	2 215	1926–30	42	.....	.....	5.7	..	31.18	37.07	10.62	73–120	0.69
Huntley	2 930	1911–30	46	.....	.....	4.3	..	30.98	42.21	8.77	89–119	0.94
Moccasin	4 228	1909–20	41	.....	.....	6.2	69	31.58	42.31	9.40	88–114	0.94
Havre	2 505	1916–20	41	44	7.8	5.7	66	33.56	42.13	8.88	89–109	0.94
Bozeman	4 754	1918–30	41	.....	.....	2.7	..	24.72	33.77	7.58	85–117	0.69
Valier	....	1916–30	41	.....	.....	3.2	..	26.60	35.89	7.74	86–112	0.69
Willow Creek Reservoir	4 130	1919–28	44	.....	.....	7.8	..	26.06	37.20	7.62	92–114	0.83
Sun River Canyon	4 474	1919–22	42	.....	.....	10.7	..	29.47	40.14	6.08	......	0.83
St. Ignatius	3 003	1922–23	45	.....	.....	....	..	26.84	36.98	5.29	86–121	0.75
(*l*) Wyoming												
Archer	6 012	1913–20	45	101	13.3	8.7	56	34.27	47.65	8.06	88–112	0.90
Sheridan	3 790	1917–20	44	47	5.5	4.3	..	33.28	43.33	9.69	89–118	0.94
Shoshone Reservoir	5 390	1916–29	46	.....	.....	....	..	31.70	43.27	9.46	86–119	0.74
Powell	4 390	1911–19	43	.....	.....	....	..	24.36	32.40	.....	......	0.76
Ralston	4 600	1911–16	44	.....	.....	....	..	31.80	40.98	.....	......	0.76
Pathfinder	5 900	1918–30	44	.....	.....	....	..	31.00	40.74	8.62	92–116	0.66
Laramie	7 150	1891–95	40	.....	.....	....	..	30.52	40.71	5.97	94–108	0.89
(*m*) Colorado												
Akron	4 650	1908–20	48	.....	.....	7.4	62	39.26	53.95	10.61	84–110	0.94 / 0.95
Lamar	3 900	1903–04	54	.....	.....	....	..	35.85	52.54	.....	......	0.83
Fort Collins	4 998	1887–1927	46	.....	6.1	1.7	68	30.41	42.19	6.12	81–119	0.80
Fort Collins	4 998	1927–28	47	.....	6.1	1.6	68	25.55	35.06	6.28	......	1.00
Denver	5 340	1916	49	.....	7.8	4.5	64	38.09	52.15	8.20	......	0.99
Pueblo	4 790	1908–09	51	.....	6.8	....	..	34.98	47.82	6.73	......	0.83
Wagonwheel Gap	9 610	1920–24	34	.....	.....	2.0	64	16.12	22.32	3.22	94–110	0.69

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity: Near-by U. S. Weather Bureau Station: Height, in feet	Wind Velocity: Near-by U. S. Weather Bureau Station: Velocity, in miles per hour	Wind Velocity: At the pan, in miles per hour	Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches: April–September	Reservoir Surface Evaporation, in Inches: Annual	Reservoir Surface Evaporation, in Inches: Maximum per month	Percentage range of annual evaporation	Coefficient for pan
(*n*) New Mexico												
Tucumcari	4 194	1913–20	57	.....	.....	6.1	50	51.51	73.56	12.89	89–114	0.94
Avalon Reservoir	3 188	1914–24, '29	63	85	7.8	....	52	42.15	60.73	10.80	90–112	0.83
Carlsbad	3 100	1910	64	.....	.....	....	..	48.11	74.01	8.87	......	0.69
Carlsbad	3 100	1910	64	.....	.....	....	..	40.60	65.10	7.65	......	0.69
Therma	8 200	1929–30	43	.....	.....	3.6	..	30.07	39.17	7.60	......	0.69
Santa Fe	7 010	1917–30	48	53	6.1	2.6	54	33.66	44.82	8.46	87–117	0.69
Farmington	5 300	1914–29	57	.....	.....	....	..	32.02	43.66	9.95	85–114	0.83
Los Griegos	4 970	1926–28	54	.....	.....	3.2	49	39.65	54.79	8.28	......	0.69
Elephant Butte	4 265	1916–30	60	.....	.....	3.9	..	47.24	66.99	11.75	87–114	0.69
Las Cruces	3 683	1918–30	60	50	6.9	1.9	41	41.15	59.46	9.38	88–114	0.69
Deming	4 300	1914–29	68	.....	.....	....	..	34.53	54.48	9.94	67–123	0.83
(*o*) Idaho												
Mud Lake	4 784	1921–23	46	.....	.....	4.4	..	30.75	39.30	7.81	......	0.83
Milner	4 200	1927–30	48	.....	.....	2.3	..	28.60	39.91	7.41	95–105	0.69
Arrowrock	3 220	1916–30	49	.....	.....	0.8	..	29.32	40.09	8.07	92–115	0.69
Aberdeen	4 400	1912–19	44	68	8.7	6.0	58	39.39	50.86	12.10	94–111	0.94
Jerome	3 893	1919–27	49	.....	.....	2.7	..	26.96	37.89	7.25	82–114	0.69
Deer Flat	2 510	1916–25	48	.....	.....	2.4	57	26.14	38.59	6.12	89–107	0.69
(*p*) Utah												
Myton	5 030	1918–30	46	.....	.....	2.9	..	33.49	43.72	9.95	93–110	0.69
Utah Lake Outlet	4 497	1901–06	50	.....	.....	....	51	32.87	40.70	9.30	......	0.80
West Portal, Strawberry Tunnel	7 500	1910–15	41	.....	.....	....	..	20.81	26.12	.....	......	0.78
East Portal, Strawberry Tunnel	7 600	1905–14, 15	36	.....	.....	....	..	15.73	19.72	.....	......	0.78
Spanish Fork	4 700	1911–15	51	.....	.....	....	55	30.23	40.10	10.47	......	0.78
Provo	4 650	1918–26	49	203	7.1	1.2	54	24.10	30.99	5.96	94–103	0.69
Provo	....	1910–17	49	203	7.1	....	..	20.81	27.29	.....	......	0.78
Utah Lake	4 497	1923–30	48	.....	.....	4.2	53	34.80	43.19	8.67	93–111	0.69
Salt Lake City	4 250	1928–30	52	203	6.2	3.6	50	40.67	50.94	10.10	95–103	0.69
Nephi	5 300	1908–27	48	.....	.....	3.7	..	41.03	50.23	11.61	88–113	0.94
Sevier Bridge Dam	4 980	1922–30	47	.....	.....	4.9	..	41.10	50.48	10.62	93–113	0.69
Piute Dam	5 900	1918–30	48	.....	.....	3.2	50	38.15	47.97	9.65	86–109	0.69
Milford	4 960	1925–27	50	43	10.0	4.3	51	46.98	59.22	10.08	......	0.99
Milford	4 960	1925–27	50	43	10.0	4.3	51	48.53	60.97	10.35	......	0.69
(*q*) Nevada												
Lamoille	.....	1918–30	45	.....	.....	2.2	..	27.88	39.45	7.35	87–111	0.69
Fallon	3 970	1908–30	51	.....	.....	3.0	52	44.50	56.74	11.10	80–112	0.94
Lahontan	4 150	1925–30	54	.....	.....	3.3	..	39.83	52.97	9.60	......	0.83
Indian Springs	3 136	1917	..	.....	.....	....	..	51.57	66.06	10.76	......	0.69
Pahrump	2 640	1920–25	62	.....	.....	1.6	..	41.97	56.29	9.24	......	0.69
(*r*) Arizona												
Roosevelt Dam	2 175	1916–30	67	.....	.....	1.4	..	41.80	56.62	10.20	81–115	0.69
Granite Reef	1 310	1910	69	.....	.....	....	..	52.63	79.51	9.84	......	0.69
Mesa	1 225	1917–30	68	107	4.9	1.5	44	37.55	53.54	8.84	86–114	0.69
Willcox	4 190	1917–30	60	.....	.....	3.0	..	40.00	60.23	9.61	82–121	0.69
Tucson	2 400	1929–30	67	.....	6.4	1.3	40	41.82	60.26	9.39	99–101	0.69
Lees Ferry	3 140	1921–30	61	.....	.....	1.9	..	46.09	60.85	10.73	......	0.69
Yuma Reservoir	127	1903	72	.....	.....	....	..	48.05	68.85	9.46	......	0.83
Yuma (Citrus)	220	1920–30	69	.....	.....	2.5	..	58.47	83.38	14.04	92–112	0.69
Yuma (Date Orchard)	127	1917–29	69	54	5.2	1.3	44	36.29	53.45	9.55	91–116	0.69
(*s*) Washington												
Lind	1 350	1926–30	50	.....	.....	4.3	59	39.64	49.76	10.20	92–107	0.85
Walla Walla	1 000	1917–30	53	65	4.9	2.0	58	29.68	36.85	8.64	88–110	0.69
Lake Kachess	2 270	1917–30	44	.....	.....	2.0	..	15.90	20.15	5.07	92–110	0.69
Yakima	1 060	1910	50	.....	.....	....	..	35.07	47.25	7.41	......	0.69
Wind River	1 000	1926–30	48	.....	.....	3.4	..	20.76	26.81	5.92	85–109	0.69
Lake Cushman	760	1924–25	50	.....	.....	....	..	18.96	21.53	4.52	......	0.83

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity: Near-by U. S. Weather Bureau Station: Height, in feet	Wind Velocity: Near-by U. S. Weather Bureau Station: Velocity, in miles per hour	Wind Velocity: At the pan, in miles per hour	Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches: April-September	Reservoir Surface Evaporation, in Inches: Annual	Reservoir Surface Evaporation, in Inches: Maximum per month	Percentage range of annual evaporation	Coefficient for pan
(*t*) **Oregon**												
Warm Springs Reservoir	....	1927-30	47	.....	.....	2.5	..	38.39	54.04	10.42	......	0.69
Burns	4 125	1914-19	44	.....	.....	4.8	59	37.72	49.05	9.57	91-105	0.94
Cold Springs	623	1914-26,'28	52	17	6.6	....	..	27.29	33.74	7.22	74-127	0.69
Hermiston	610	1912-23	52	.....	.....	3.0	..	33.83	41.31	.....	......	0.94
Hermiston	600	1909-13	52	17	7.9	....	..	41.88	51.41	.....	......	0.83
Moro	1 800	1911-19	48	.....	.....	6.8	..	39.53	54.51	10.86	92-108	0.94
Merrill	4 070	1913-19	46	.....	.....	....	..	28.66	37.90	.....	......	0.83
Klamath Falls	4 100	1924-29	48	.....	.....	....	..	24.08	36.71	6.33	......	0.83
Keno	4 090	1904-09	48	.....	.....	....	..	24.82	30.05	6.65	......	0.83
Ady	4 090	1910	48	.....	.....	....	..	35.54	43.26	7.06	......	0.83
Corvallis	235	1922-30	52	.....	.....	1.5	..	21.88	30.68	5.92	91-106	0.69
(*u*) **California**												
Tahoe	6 230	1916-30	42	.....	.....	3.0	..	22.34	32.09	6.67	84-110	0.88
Lake Elsinore	4 700	1910-18	51	.....	.....	....	..	26.12	35.82	8.22	77-132	0.83
Clear Lake	4 350	1911-13	44	.....	.....	....	..	26.95	35.54	.....	......	0.83
Coppocks	4 035	1916-29	46	.....	.....	....	..	29.38	38.15	8.02	78-120	0.83
Fall River Mills	3 340	1925-30	50	.....	.....	2.3	..	37.35	47.65	9.07	94-108	0.63
Crane Valley Lake	3 300	1912-20	56	.....	.....	....	..	32.20	44.89	8.84	82-110	0.89
Independence	3 766	1909-11	56	28	6.6	....	45	41.15	55.61	8.62	......	0.83
Independence	3 800	1909-11	56	28	6.6	....	45	38.64	55.26	8.00	......	0.83
Cuyamaca Lake	4 640	1913-15	52	.....	.....	....	..	39.26	58.20	9.71	......	0.73
Cuyamaca Lake	4 620	1913-15	52	.....	.....	....	..	38.75	62.25	8.95	94-106	0.88
East Park Reservoir	1 200	1911-29	59	.....	.....	4.0	..	39.48	50.84	9.51	82-117	0.83
Warner Hot Springs	2 780	1913-15	53	.....	.....	....	..	34.64	52.94	8.67	96-108	0.83
San Luis Rey Creek	2 620	1913-15	56	.....	.....	....	..	29.29	45.77	8.67	83-116	0.83
Dodgeland	160	1919-22	61	.....	.....	2.0	..	34.09	42.06	8.65	90-112	0.69
Biggs	94	1913-19	72	.....	.....	3.8	..	41.40	56.09	11.24	91-107	0.94
Oakdale	215	1918-30	60	.....	.....	5.2	..	45.75	56.89	11.76	92-116	0.69
Kingsburg	285	1881-85	63	.....	.....	....	..	28.60	38.38	.....	......	0.83
La Mesa Reservoir	480	1913-15	66	.....	.....	....	..	36.94	55.46	8.01	......	0.83
Sweetwater Reservoir	150	1889-93	62	.....	.....	....	71	32.54	48.55	7.52	......	0.83
Alvarado	3	1924-30	57	.....	.....	4.8	52	26.79	37.58	5.83	91-106	0.69
Chula Vista	9	1918-30	59	.....	.....	3.8	..	26.84	42.26	5.45	89-108	0.69
Indio	-15	1910	73	.....	.....	3.3	33	64.87	89.49	12.26	......	0.75
Brawley	-113	1910	71	.....	.....	4.1	37	55.31	77.66	10.60	......	0.75
Mecca	-189	1910	72	.....	.....	4.7	38	57.41	80.87	11.41	......	0.75
Near Salton Sea	-230	1910	73	.....	.....	4.1	40	66.35	97.10	13.10	......	0.59
(*v*) **Canada**												
Keetwatin, Ont	....	1913-28	36	175	6.0	....	72	11.24	14.29	3.84	82-117	0.83
Saskatoon, Sask	....	1918-30	..	.....	.....	....	..	23.51	28.89	7.14	84-135	0.84
Edmonton, Alta	....	1918,'21-22	..	.....	.....	....	..	18.65	24.00	5.46	......	0.84
Lake Newell, Alta	....	1919-25	39	.....	.....	2.3	..	26.37	32.32	7.37	......	0.84
Strathmore, Alta	....	1915	..	.....	.....	....	..	22.09	27.94	5.44	......	0.84
Coaldale, Alta	....	1915-21	39	.....	.....	....	..	25.82	32.42	6.82	......	0.84
(*w*) **Caribbean Region**												
San Juan, Puerto Rico	82	1919-30	78	.....	11.8	....	78	31.34	55.29	6.54	89-107	0.66
Christiansted, Virgin Islands	20	1919-30	79	.....	.....	....	..	28.45	49.88	6.44	91-114	0.69
Gatun, Canal Zone	85	1912-30	80	.....	.....	7.1	84	22.47	48.38	7.03	78-112	0.83
Pedro Miguel, Canal Zone	55	1919-30	79	.....	.....	5.2	83	19.46	42.59	6.14	88-108	0.83
Gamboa, Canal Zone	85	1918-30	79	.....	.....	5.2	..	16.59	36.76	5.66	82-118	0.83
Alhajuela, Canal Zone	120	1918-30	78	.....	.....	1.8	..	18.66	43.04	6.75	92-108	0.83
(*x*) **Hawaii**												
Maunawili, Oahu	250	1921-30	72	.....	.....	1.9	..	17.23	30.92	3.80	94-108	0.69
Upper Hoaeae, Oahu	705	1920-30	72	.....	.....	1.2	..	25.74	42.58	5.68	93-112	0.69

TABLE 8.—(*Continued*)

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity			Relative humidity (percentage)	Reservoir Surface Evaporation, in Inches			Percentage range of annual evaporation	Coefficient for pan
				Near-by U. S. Weather Bureau Station		At the pan, in miles per hour		April-September	Annual	Maximum per month		
				Height, in feet	Velocity, in miles per hour							
(y) Egypt and the Sudan												
Alexandria	....	1920-29	68	.....	5.2	....	72	.....	28.76	.....	......	0.50
Giza (Cairo)	....	1920-27	67	.....	5.2	....	69	.....	40.26	.....	......	0.50
Asyut	....	1920-29	71	.....	5.2	....	54	.....	64.71	.....	......	0.50
Wadi Halfa	....	1905-29	76	.....	5.2	....	32	.....	113.60	.....	......	0.50
Merowe	....	1905-29	83	.....	5.6	....	22	.....	120.79	.....	......	0.50
Atbara	....	1905-29	83	.....	5.6	....	29	.....	123.67	.....	......	0.50
Khartoum	....	1905-29	83	.....	5.6	....	31	.....	107.85	.....	......	0.50
Roseires	....	1905-29	79	.....	5.6	....	56	.....	76.21	.....	......	0.50
Malakal	....	1915-29	79	.....	2.8	....	58	.....	64.71	.....	......	0.50
Wau	....	1906-29	79	.....	2.8	....	63	.....	53.21	.....	......	0.50
Mongalla	....	1906-29	79	.....	2.8	....	70	.....	43.14	.....	......	0.50
(z) Turkestan												
Khodjent	1 100	1920-29	..	.....	.....	....	..	33.76	42.84	.....	......	0.95
Fergana	1 300	1920-29	..	.....	.....	....	..	29.25	38.05	.....	......	0.95
Andidjan	1 500	1920-29	..	.....	.....	....	..	29.83	37.99	.....	......	0.95
Osh	3 500	1920-29	..	.....	.....	....	..	25.88	33.96	.....	......	0.95

records have been reduced to the common basis of evaporation from reservoir surfaces, and this fact should be considered in studying the figures given in the accompanying tables.

The records, which were taken by pans for which the coefficients had been determined, have been expanded to the full year where necessary. Where the meteorological data were available, the evaporation for the missing periods (usually the winter months) has been either computed or estimated.

The summary of these records, Table 8, is arranged in geographic order, beginning with the northeastern part of the United States. Following the records of the United States are those for outlying possessions and foreign countries (See Table 8(*v*) to (*z*)).

Omitted from the summary are the records at Lawrence, Boston, and Fall River, Mass., and Rochester, N. Y., which were taken from wooden or fiber tanks, for which no coefficients for reduction to reservoir surface evaporation are available (see Table 9). Of these, the best known are the Boston records, taken by the late Desmond FitzGerald, Past-President and Hon. M. Am. Soc. C. E. From 1876 to 1884, these records were taken by observations on small wooden tanks floating in the Chestnut Hill Reservoir, and as wood does not transmit heat readily, it was found that at times the temperature in the tanks varied as much as 10° F from that of the water in the reservoir itself. The same condition persisted when a large wooden tank, 10 ft in diameter and 10 ft deep, was installed in 1884.[22] A summary of these records as observed is given (Table 8), together with others which can not be reduced to reservoir surface evaporation at this time.

[22] *Transactions*, Am. Soc. C. E., Vol. 15 (1886), p. 596.

Effect of Factors Influencing Evaporation

The principal factors influencing evaporation are so interdependent that it is difficult to segregate the effect of any one. The relative effect of the variation in any one factor can be shown by selecting, from the many available records, pairs in which two factors are practically the same and the third is widely different. The difference in evaporation may then be considered a measure of the effect of varying the third factor. Table 10 shows the relative effect of each factor.

TABLE 9.—Summary of Evaporation Records for Which no Reduction Factors Are Available

Station	Elevation, in feet	Years	Temperature of air, in degrees Fahrenheit	Wind Velocity			Relative humidity (percentage)	Observed Evaporation, in Inches			Percentage range of annual evaporation
				Near-by Weather Bureau Station		At the pan, in miles per hour		April-September	Annual	Maximum per month	
				Height, in feet	Velocity, in miles per hour						
Lawrence, Mass	100	1888–90	45	.....	.....	....	..	.....	29.16	5.20	87–119
Boston, Mass	135	1876–86	49	.....	.....	10.2	72	.....	39.11	.....	....
Fall River, Mass	130	1899–1901	50	.....	.....	....	72	.....	38.71	6.74	96–105
Rochester, N. Y	620	1891–1925	50	.....	8.5	6.0	72	.....	31.78	6.86	82–123
Charleston, S. C	25	1905–23	66	.....	10.0	....	79	29.77	47.04	.....	....
Albuquerque, N. Mex	5 000	1900, 03–04	56	.....	8.0	....	..	.....	70.01	12.63	....
Bumping Lake, Wash	3 400	1912	41	.....	.....	....	..	28.23	32.48	8.26	....
Little Bear Valley, Calif	5 200	1896–97	51	.....	4.9	....	..	29.30	36.91	6.60	....
Kingston, Jamaica	100	1924–30	79	.....	7.6	....	79	.....	74.00	9.44	....

Variation of Evaporation in the United States

Most of the records of evaporation were taken in the western part of the United States, comparatively few being available east of the Mississippi River. These records indicate in a general way, however, the variation in the United States.

The area of lowest evaporation is the Great Lakes region, where it ranges from 15 to 20 in. per year. East of the Mississippi River the evaporation increases from 20 in. in Maine to 43 in. at Birmingham, Ala., and then decreases slightly toward the Gulf Coast, with its greater humidity.

West of the Great Lakes the evaporation increases to 40 in. in the Upper Missouri River Basin. Southwest of the Missouri River, it gradually increases to 70 in. in Southwestern Texas and Southeastern New Mexico.

In the Rocky Mountain region, the evaporation depends largely upon temperature, which decreases with an increase in elevation. As an example of this fact, the records in Table 11 are cited.

As few records are available at the higher elevations, it is impossible to estimate the variation in evaporation in the Rocky Mountain region.

In the inter-mountain region, between the Rockies and the Sierra Nevada, the evaporation ranges from 38 in. in Northern Nevada to 60 in. in Southern Nevada and Arizona.

TABLE 10.—RELATIVE EFFECTS OF TEMPERATURE, WIND VELOCITY, AND HUMIDITY ON EVAPORATION

Station	Temperature, in degrees Fahrenheit	Wind at pan, in miles per hour	Relative humidity (percentage)	Evaporation from reservoir surfaces, in inches
(*a*) RELATIVE EFFECT OF TEMPERATURE				
Washington, D. C.	54	2.3	69	34.53
Austin, Tex.	68	2.0	68	45.53
Difference	14	0.3	1	11.00
Denver, Colo.	49	4.5	64	52.15
Laredo, Tex.	73	3.8	64	67.47
Difference	24	0.7	0	15.32
Wagonwheel Gap, Colo.	34	2.0	64	22.32
Wau, Sudan	79	2.8	63	53.21
Difference	45	0.8	1	30.89
(*b*) RELATIVE EFFECT OF WIND VELOCITY				
Columbia, Mo.	54	1.5	71	28.13
Columbia, Mo.	54	2.9	71	35.82
Difference	0	1.4	0	7.69
Silverhill, Ala.	67	1.8	78	38.27
Crowley, La.	68	2.8	77	45.92
Difference	1	1.0	1	7.65
Provo, Utah	49	1.2	54	30.99
Utah Lake Outlet	48	4.2	53	43.19
Difference	1	3.0	1	12.20
Pedro Miguel, Canal Zone	79	5.2	83	45.11
Gatun, Canal Zone	80	7.1	84	51.47
Difference	1	1.9	1	6.36
(*c*) RELATIVE EFFECT OF RELATIVE HUMIDITY				
Beeville, Tex.	70	5.7	81	54.94
Brawley, Calif.	71	4.1	37	77.66
Difference	1	1.6	44	22.72
Chapel Hill, N. C.	61	1.1	69	28.56
Las Cruces, N. Mex.	60	1.9	41	58.46
Difference	1	0.8	28	30.90
Pedro Miguel, Canal Zone	79	5.2	83	42.59
Roseires, Egypt	79	5.6	56	76.21
Difference	0	0.4	27	33.62
Malakal, Sudan	79	2.8	58	64.71
Mongalla, Sudan	79	2.8	70	43.14
Difference	0	0	12	21.57

In the arid section of Washington and Oregon, comprising chiefly the region east of the Coast Range, evaporation ranges from 40 to 50 in. In the western half of Washington, the evaporation is generally less than 25 in., the lowest being at Lake Kachess, with a rate of 20.15 in. In Western Oregon, it is between 30 and 40 in.

In California, the rate of evaporation depends largely upon the elevation of the station. In the Sierra Nevada it ranges from 32 in. at Lake Tahoe

TABLE 11.—EFFECT OF TEMPERATURE AND ELEVATION ON EVAPORATION

Station	Elevation, in feet	Temperature, in degrees Fahrenheit	Evaporation, in inches
Wagonwheel Gap, Colo.	9 610	34	22.32
Therma, N. Mex.	8 200	43	39.17
East portal, Strawberry Tunnel, Utah	7 600	36	19.72
West portal, Strawberry Tunnel, Utah	7 500	41	26.12
Santa Fe, N. Mex.	7 010	48	44.82

to 62 in. in the southern part of the State. Throughout the remainder of California it ranges from 38 to 57 in., except in Imperial Valley, where an extreme of 97.10 in. has been recorded near Salton Sea, in the center of the valley.

STANDARD EQUIPMENT FOR EVAPORATION STATIONS

FINAL REPORT OF SUB-COMMITTEE ON EVAPORATION OF THE SPECIAL COMMITTEE ON IRRIGATION HYDRAULICS

The recommendations of the Special Committee on Irrigation Hydraulics regarding standard equipment at evaporation stations, operated primarily to determine the evaporation from large water surfaces in the immediate neighborhood, are set forth as follows.

Pans

Two or more pans are highly desirable. If only one is feasible, the Class *A* land pan is preferred. If more than one is justified, the order of preference is as follows:

(a) United States Weather Bureau Standard Class *A* evaporation (land) pan. (A round land pan; 4 ft in diameter and 10 in. deep; of 22-gauge galvanized iron, without paint; supported on grillage; bottom of pan is 6 in. above ground surface. Details of set-up and procedure may be found in "Instructions for Installation and Operation of Class *A* Evaporation Stations," U. S. Weather Bureau, *Bulletin 559, Circular L,* Instrument Division.)

(*b*) Modified Colorado "buried" pan. (A square land pan; 3 ft on side; 18 in. depth; sunk in ground to within 4 in. of top; made of 18-gauge galvanized iron, without paint.)

(*c*) United States Geological Survey floating pan. (A square floating pan; same dimensions and material as in Preference (*b*); supported by two cylindrical metal floats, each 9 in. in diameter and 42 in. long; tank floating with 3 in. of its rim above the water surface.)

Essential Equipment

Two items of equipment may be considered essential, namely:

One gauge for measuring evaporation.

One standard Weather Bureau rain gauge.

Auxiliary Equipment

The following auxiliary equipment is listed in order of importance:

(*a*) One anemometer for recording wind velocities.

(*b*) One set of maximum-and-minimum thermometers, for air temperatures, housed in the standard instrument shelter of the U. S. Weather Bureau.

(*c*) One set of maximum-and-minimum thermometers for water temperatures.

(*d*) One sling psychrometer, to determine the relative humidity of air.

Alternate Equipment.—The following equipment may be substituted under the conditions stated:

(*b*′) Alternate to (*b*); one ordinary thermometer for air temperatures. Where readings can be taken at 12-hr intervals.

(*c*′) Alternate to (*c*); one ordinary thermometer for water temperatures. Where readings can be taken at 12-hr intervals.

(*b*″) Alternate to (*b*) or (*b*′); one thermograph for continuous record of air temperatures.

(*d*′) Alternate to (*d*); one recording hair-hygrometer for continuous record of humidity.

Pan Coefficients

Greater depth of water is evaporated from pans than from reservoir surfaces; hence a coefficient is necessary to convert pan evaporation to the equivalent evaporation from a reservoir. Table 12 gives type of pan, recommended coefficient, and observed range of this coefficient.

TABLE 12.—Recommended Coefficients and Observed Range

Type of pan	Coefficient	Reasonable range of coefficient
(*a*) Class *A* land pan	0.70	0.60 to 0.82
(*b*) Modified Colorado "buried" pan	0.78	0.75 to 0.86
(*c*) U. S. Geological Survey floating pan	0.80	0.70 to 0.82

Reasons for Selection of Class *A* Land Pan as Standard

The first selection must be made as between land pans and floating pans. The Sub-Committee reports two arguments in favor and four against the use of floating pans.

In favor of floating pans are the following:

(1) The amount of evaporation from the pan more nearly approximates that from the surrounding water; that is, a coefficient nearer 1.00 can be used.

(2) In controversial matters, the observations on a floating pan would be more acceptable to many engineers than those from a land pan.

Against a floating pan are the following:

(1) The ever-present possibility of the pan shipping water from the reservoir or losing water due to pitching of waves.

(2) A question as to the effect of measures to prevent shipping or loss of water. The same measure that retards gain or loss of water may set up a condition that increases or diminishes the actual evaporation loss in the pan, compared with that in the reservoir.

(3) Comparative difficulty of access may lead to poor attendance and to "desk readings."

(4) In controversial matters, records are difficult of unquestionable proof.

The items against the floating pan were determined to outweigh those in its favor so the land pan was preferred. The specific reasons are as follows: The U. S. Weather Bureau Standard Class *A* evaporation pan is selected as first choice because it has more features in its favor and fewer unsatisfactory ones than any other type of pan now known or thought obtainable by new design. It does not approach the ideal. Other pans excel it in certain respects, but are far behind it in others.

A summary for and against the U. S. Weather Bureau, Class *A* pan is as follows:

The advantages are:

(1) More data on this pan are available for comparison than on any other particular type of pan.

(2) The coefficient has been found to be about the same for this pan in comparison with large water surfaces in many locations and under many conditions.

(3) It is easy of access for observations.

(4) It is not subject to uncertainty due to inward or outward wash of water as is the case for a floating pan.

(5) It is raised above reasonable drift of dirt, débris, and snow.

(6) It is reasonable in cost of installation.

The disadvantages are:

(1) The coefficient is considerably less than unity; that is, the rate of evaporation is much greater from the pan than from a reservoir surface.

(2) For the data available, coefficients for certain months vary greatly from those for other months. This inconsistency, however, is true for all types of pans thus far studied.

By Sub-Committee on Evaporation,

FRED C. SCOBEY, *Chairman*,
IVAN E. HOUK,
R. L. PARSHALL,

June 18, 1932. CARL ROHWER, *Co-Operating Member*.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

GEORGE WASHINGTON BRIDGE MATERIALS AND FABRICATION OF STEEL STRUCTURE

By Herbert J. Baker,[1] Esq.

Synopsis

The main structure of the George Washington Bridge, because of its type and magnitude, required for its construction large quantities of metallic materials of various kinds.

To describe the methods of manufacture and fabrication followed in the mills and shops on the various parts of the metallic structure, to record the properties of the materials incorporated in the structure, and to outline the methods of testing that were followed in order to insure conformity with the specifications are the objects of this paper.

No material in this structure is new and untried. Except for modifications in the requirements for yield point of silicon steel in the towers and for tensile strength and yield point for heat-treated anchorage eye-bars and cable wire, like materials have been manufactured in the past to similar specifications.

The most modern methods of manufacture were followed in the various operations. These operations are of special interest because of the large quantities of materials involved and because of the unusual size of members.

Introduction

The four principal divisions of the main steel superstructure, listed in the order of their construction, are the anchorages, towers, cables, and suspended structure. Each of these divisions contains a number of different kinds of material.

In the anchorages are structural steel girders to which the cables are connected by chains of heat-treated eye-bars. The lower links of eye-bars are connected to the girders by heat-treated carbon-steel pins. The remaining pins of the eye-bar chains are made of annealed carbon steel. The cables are connected to the eye-bars by means of cast-steel strand shoes.

Note.—Discussion on this paper will be closed in the May, 1933, *Proceedings*.

[1] Engr. of Inspection, The Port of New York Authority, New York, N. Y.

The towers are made of rolled structural steel. The cables are constructed of cold-drawn wire. They are supported at the towers and at the turning points on the anchorages by cast-steel saddles. The cables are wrapped with soft galvanized wire at points not otherwise covered. The tower and anchorage saddles rest on annealed carbon-steel rollers and rockers, respectively. The cable bands, which are made of cast steel, are clamped to the cables by heat-treated carbon-steel bolts.

Wire ropes, to which are attached cast-steel sockets, are the means by which the floor system is suspended from the cables. The floor system is made of rolled structural steel. In this part of the structure are miscellaneous pins of annealed carbon steel; miscellaneous cast-steel members; cast phosphor-bronze bushings; and rolled manganese-bronze wearing plates.

The materials are treated under the following headings: (1) Heat-Treated Eye-Bars; (2) Structural Steel; (3) Cast Steel; (4) Cable Wire; (5) Suspender Rope; (6) Other Materials; and (7) Distribution of Materials in Main Divisions of Structure.

Materials were inspected in various stages of manufacture. For this work, a force of men, skilled in the inspection of the various kinds of materials, was organized. In addition to the inspection at the place of manufacture, samples were analyzed by the Port Authority in its laboratory. For the analytical work a Chief Chemist, experienced in analyzing metallic materials, and the necessary assistants were employed. The personnel (a maximum of forty men) engaged in inspection and analytical work on this and other Port Authority projects was under the direction of the writer.

All physical tests upon which the acceptance of the materials was based were made by the manufacturers at their plants. These tests were witnessed by the Port Authority inspectors. In addition to the acceptance tests, a number of "check" tests were made by the Port Authority in its laboratory as special studies. The manufacturers were required by the specifications to furnish the check test specimens, machined and ready for testing.

(1) Heat-Treated Eye-Bars

Eye-bars are used only in the anchorages of the structure. They are 10 in. wide, but variable in the other two dimensions, being $1\frac{5}{16}$ in., $1\frac{3}{4}$ in., or $1\frac{7}{8}$ in. thick and from 35 ft $3\frac{13}{32}$ in. to 38 ft $7\frac{27}{32}$ in., between centers of pin-holes. For the two anchorages 3 456 bars are used.

The function of the anchorage eye-bars and the considerations governing the choice of material are treated in the paper on the design of the superstructure.[2] A heat-treated material with minimum values for yield point and tensile strength of 50 000 and 80 000 lb per sq in., respectively, and a total elongation of at least 8% in 18 ft, was specified. It was further stipulated that any twelve consecutive tests should show a minimum average yield point of 53 300 and a minimum average tensile strength of 85 000 lb per sq in., respectively.

[2] "George Washington Bridge: Design of Superstructure," by Allston Dana and Aksel Andersen, Members, Am. Soc. C. E., and George M. Rapp, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., December, 1932, p. 1681.

Making and Rolling the Steel.—The steel was made and rolled into flats, of the width and thickness desired in the eye-bars, at the Homestead, Pa., plant of the Carnegie Steel Company. The only values specified for the untreated material were for the chemical properties of phosphorus and sulfur, the content of which was not to exceed 0.04% and 0.05%, respectively. The material furnished was a carbon steel of the following average analysis: Carbon, 0.35%; manganese, 0.60%; phosphorus, 0.022%; and sulfur, 0.035 per cent. It was made by the basic open-hearth process.

All ingots were made by pouring the metal in the top of the moulds. In some cases hot top moulds were used. The ingots, in general, had a cross-section of 27 by 32 in. and were bloomed to one of 16 by 16 in. In making the blooming-mill discard care was exercised to eliminate piping and harmful segregation. Slabs of two sizes were cut from the bloom from which were rolled either a single-length or a double-length eye-bar flat.

The slabs were rolled to eye-bar flats on a 42-in. Universal plate mill. This mill is equipped with a manipulator for turning the slab on edge as all rolling is done with the horizontal rolls until the slab has been reduced to a square section, each side of which is slightly greater than the width of the required eye-bar flat. The side or vertical rolls are then brought into play to obtain and maintain the desired width, and rolling is continued until the desired thickness is obtained. It should be noted that the 27 by 32-in. ingot was reduced, by rolling, about 98% in producing an eye-bar flat.

Physical properties of the material in the "as-rolled" condition were not specified. However, tensile tests on material from each melt met certain minimum standards set by the eye-bar manufacturer, namely, yield point and tensile strength of 35 000 and 70 000 lb per sq in., respectively, and an elongation of 20% in 8 in.

Manufacture of Eye-Bars.—The rolled eye-bar flats were shipped from the mill to the Ambridge, Pa., plant of the American Bridge Company for manufacture into eye-bars.

In the order of their occurrence, the mechanical operations conducted in manufacturing these bars consisted of forming the heads before heat treatment and of straightening the bars and boring the pin-holes after heat treatment.

The heads were formed by repeating the operations of upsetting and rolling until the correct shape and thickness had been attained. After completing this operation, a pin-hole about $\frac{3}{4}$ in. smaller than the bored diameter was punched in the head.

The heat-treating furnaces for hardening and drawing the bars were gas-fired and equipped with pyrometers. Live rolls were used to carry the bars into and through the furnaces. The bars were hardened one at a time, the quenching medium being water, in which they were immersed edgewise. The bars were drawn two at a time, and when removed from the furnace were allowed to cool, under natural conditions, in shop air.

The straightening operation was a major one as the bars were kinked and cambered edgewise and flatwise and warped by the heat-treating opera-

tions to such an extent that they all required straightening. Edgewise camber of 4 to 6 in. in 24 ft occurred frequently. The straightening operation was performed on cold bars by means of gag-press and rolls.

Scope of Tests.—Following the usual practice in regard to proving the quality of eye-bars, tests to destruction of full-sized heat-treated members were specified. The number of full-sized tests had to be approximately 5% of the number of bars made in the early stages of manufacture and had to average about 3% of the number required for the work. In addition to the full-sized test of an occasional bar, the Brinell hardness test was required to be made on every bar. Specimen tests were also made on heat-treated material, the specimens for which were machined from eye-bar flats.

In addition to the data secured from the aforementioned tests, it was desired to determine the stress-strain characteristics of full-sized eye-bars. Therefore, tests to destruction were performed on two full-sized members in such manner as to obtain this information.

Brinell Hardness Test.—The need for a test to indicate the strength of each bar may be appreciated when the method of heat treatment is considered. The bars were heated and quenched one at a time; they were drawn two at a time. Obviously, a full-sized test does not truly represent the bars in any lot, nor does it indicate particularly a possible inclusion of other grades of material. The hardness test was used to overcome this deficiency.

The test conformed to Specifications of the American Society for Testing Materials entitled "Tentative Methods of Brinell Hardness Testing of Metallic Materials (Serial Designation E 10-27)." Accordingly, after the bar had cooled to shop temperature, a load of 3 000 kg was applied for 20 sec through a hardened steel ball 10 mm in diameter. As a rule, the impression was made at the approximate center line in three places—at the middle and near each head—on one side of those bars not used for full-sized tests, but at intervals of about 2 ft on both sides of the test bars.

The results of the Brinell hardness test were to be considered merely as a rough measure of the strength of the individual bars. However, the full-sized tests indicated that a range of average hardness numbers between 183 and 212, inclusive, could be considered as a reliable indication of the physical properties desired in the eye-bars. The Brinell hardness testing machine was not calibrated for this work because it was used only as a means of comparing the hardness of similar members. This fact should be borne in mind when considering the hardness numbers referred to herein.

Full-Sized Acceptance Tests.—To gauge the quality of the eye-bars not tested to destruction is, of course, the purpose of the full-sized test. Consequently, the selection of the bar to serve as a test specimen becomes a matter of prime importance. The choice should necessarily be governed by the history of the bar in order to acquire results indicative of the quality of other members showing approximately the same record as to chemical properties and details of manufacture. When, by some means, the indicated quality of the bar not tested to destruction is confirmed, then its quality can be readily accepted.

In this work, the bar's identity, a detailed record of its manufacture, and its Brinell hardness were available. To make the hardness test a dependable means for measuring the strength of an eye-bar or, in other words, to ascertain the relation between the hardness and other physical properties in order to estimate the strength of other bars, specimens differing from one another in average hardness, and others showing wide range in hardness within themselves, were tested to destruction.

Instead, therefore, of selecting bars for use merely as samples of lots containing a certain number of bars (in other words, at random, as was specified), they were chosen generally with the view of investigating the effects, on the physical properties, of a number of variable factors that were noted in the manufacture from steel mill to finished product. A list of these factors would include the chemical properties; position that the bar occupied in the ingot (as top, center, or bottom); surface imperfections (as scores or guide marks); quenching or drawing temperatures, or a combination of both; re-quenching; re-drawing; re-forging followed by re-treating; hardness numbers and uniformity of hardness throughout length; and excessive straightening, particularly removal of short local kinks or twists. The effects of one of these factors (chemical properties), are predictable and could be confirmed by the Brinell hardness test supported by data gathered from test bars as the manufacturing progressed. The extent to which other factors affected the desired physical properties of the bars could be ascertained only by a full-sized test.

A total of 111 full-sized tests were made. Two of these tests gave results outside the specified requirements, the elongation in 18 ft being about 50% of the minimum specified. The records of the two test bars disclosed nothing that would indicate to what these failures might be attributed. The average results of the remaining 109 tests are given in Table 1. All specimens broke in the body of the bar.

TABLE 1.—RESULTS OF 109 FULL-SIZED EYE-BAR TESTS

	Minimum	Average	Maximum
Yield point, in pounds per square inch	50 900	57 700	63 300
Tensile strength, in pounds per square inch	80 200	89 300	97 700
Percentage elongation, in 18 ft	8.5	10.8	13.9
Percentage reduction of area	25.6	45.4	54.5

The full-sized members were tested in an hydraulic type of testing machine at the Ambridge plant of the American Bridge Company. This apparatus was calibrated against the Emery testing machine at the National Bureau of Standards, U. S. Department of Commerce, by the use of two 10 by $1\frac{3}{4}$ in. by 16-ft eye-bars. The results of the tests indicated that for the same elongation the eye-bar testing machine loads were slightly higher than those of the Emery testing machine at the Bureau of Standards, but the error has been given no consideration in the recorded results of full-sized eye-bar tests.

The two bars used for calibration purposes were tested later to destruction in the eye-bar testing machine. The results and description of this test are given subsequently under the heading, "Stress-Strain Test of Full-Sized Bars."

Specimen Tests.—Studies were made, by means of specimen tests, to determine the uniformity in physical properties of the material throughout the cross-section of heat-treated eye-bar flats.

The specimen material, two 12-ft flats, one from each of two open-hearth melts, was heat-treated in the same manner and with the same equipment as the eye-bars. The axes of the specimens were parallel with the longitudinal axes of the flats, and from the 10-in. width of each flat, nine specimens, equally spaced, were obtained. The specimens were machined to a diameter of 0.505 in. with a gauge length of 2 in.

The average tensile strength and yield point of the material (as revealed by the tests of the two specimens machined from the edges of one flat) were 103 000 and 67 500 lb per sq in., respectively. The corresponding average for the remaining seven specimens from this flat were 85 400 and 50 800, ranging between 83 700 and 88 000 for tensile strength and between 49 000 and 53 000 for yield point. The tests of the specimens from the other flat, while differing somewhat in absolute values, showed very similar variations in physical properties between the material at the edge of the flat and in the body of the cross-section. The average modulus of elasticity of the material, all tests considered, was found to be 29 900 000 lb per sq in., ranging between 28 600 000 and 30 300 000 lb per sq in., which corresponds with the value of 29 700 000 lb per sq in. for this property obtained on the tests of full-sized bars.

Stress-Strain Tests of Full-Sized Bars.—The results of special tests made for the purpose of ascertaining the characteristics of full-sized heat-treated eye-bars under tension are plotted in Fig. 1, and arranged for comparison in Table 2.

TABLE 2.—LOADS AND ELONGATIONS, FULL-SIZED, HEAT-TREATED EYE-BARS UNDER TENSION (SEE FIG. 1).

Load, in thousands of pounds	Total Inches of Elongation,* in a Gauge Length of 100 Inches		Load, in thousands of pounds	Total Inches of Elongation,* in a Gauge Length of 100 Inches		Load, in thousands of pounds	Total Inches of Elongation,* in a Gauge Length of 100 Inches	
	Test 1055*A*	Test 1055*B*		Test 1055*A*	Test 1055*B*		Test 1055*A*	Test 1055*B*
DIAL READINGS			975....	0.1731	0.1770	1 275....	1.825	2.205
			1 000....	0.1823	0.1855	1 300....	2.025	2.405
100...	0.0000	0.0000	1 025....	0.1909	0.1965	1 325....	2.225	2.670
200...	0.0184	0.0186	1 050....	0.2052	0.2115	1 350....	2.415	2.915
300...	0.0386	0.0372	1 075....	0.2404	....	1 375....	2.635	3.170
400...	0.0568	0.0563	1 100....	0.4879	....	1 400....	2.900	3.445
500...	0.0754	0.0750	1 105....	0.7504	....	1 425....	3.150	3.760
550...	0.0852	0.0853	1 105....	0.7694	....	1 450....	3.450	4.170
600...	0.0942	0.0942				1 475....	3.800	4.620
650...	0.1038	0.1039	SCALE READINGS			1 500....	4.250	5.170
700...	0.1134	0.1128				1 525....	4.675	5.870
750...	0.1220	0.1223	1 075....	....	0.895	1 550....	5.350	7.020
775...	....	0.1262	1 100....	....	0.980	1 570....	....	8.770
800...	0.1328	0.1313	1 105....	0.820	....	1 575....	6.150	....
825...	....	0.1364	1 125....	0.840	1.155	1 600....	7 350	....
850...	0.1438	0.1411	1 150....	0.985	1.295	1 620....	10 000	....
875...	....	0.1467	1 175....	1.175	1.490	....	....	....
900...	0.1544	0.1530	1 200....	1.325	1.655	....	....	....
925...	0.1604	0.1578	1 225....	1.475	1.805	....	....	....
950...	0.1668	0.1650	1 250....	1.660	1.990	....	....	....

* Average readings top and bottom.

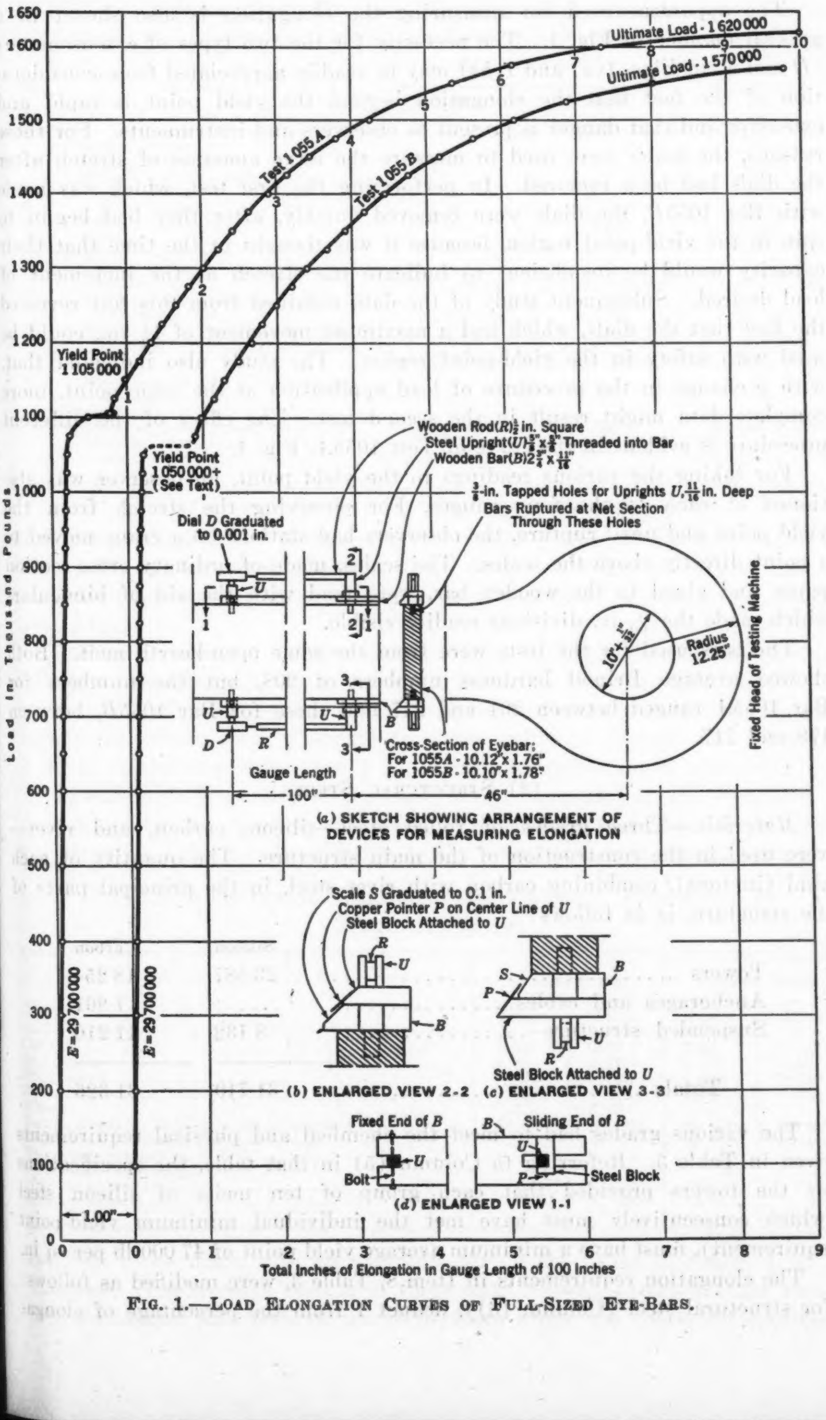


FIG. 1.—LOAD ELONGATION CURVES OF FULL-SIZED EYE-BARS.

The apparatus used for measuring the elongation is also shown in a general manner in Fig. 1. The necessity for the two types of extensometers (D and S, in Figs. 1(a) and 1(b)) may be readily appreciated from consideration of the fact that the elongation beyond the yield point is rapid and extensive and that danger is present to observers and instruments. For these reasons, the scales were used to measure the large amounts of stretch after the dials had been removed. In performing the first test, which was made with Bar 1055B, the dials were removed quickly, after they had begun to spin in the yield-point region, because it was thought at the time that their capacity would be insufficient to indicate the stretch at the increment of load desired. Subsequent study of the data obtained from this test revealed the fact that the dials, which had a maximum movement of $1\frac{1}{2}$ in., could be used with safety in the yield-point region. The study also indicated that, with a change in the procedure of load application at the same point, more complete data might result in the second test. The effect of the different procedure is evident in the curve of Test 1055A, Fig. 1.

For taking the various readings to the yield point, an observer was stationed at each of the four gauges. For observing the stretch from the yield point and until rupture, the observers had stations on a crane moved to a point directly above the scales. The scales, made of ordinary cross-section paper and glued to the wooden bar, were read with the aid of binoculars which made the $\frac{1}{10}$-in. divisions readily visible.

The bars used for the tests were from the same open-hearth melt. Both showed average Brinell hardness numbers of 208, but the numbers for Bar 1055A ranged between 201 and 217 and those for Bar 1055B, between 179 and 217.

(2) Structural Steel

Materials.—Three grades of rolled steel—silicon, carbon, and rivet—were used in the construction of the main structure. The quantity of each kind (in tons), combining carbon with rivet steel, in the principal parts of the structure, is as follows:

	Silicon	Carbon
Towers	23 587	18 254
Anchorages and cables	......	1 861
Suspended structure	8 132	11 210
Totals	31 719	31 325

The various grades had to meet the chemical and physical requirements given in Table 3. Referring to Column (5) in that table, the specifications for the towers provided that each group of ten melts of silicon steel (which consecutively must have met the individual minimum yield-point requirement), must have a minimum average yield point of 47 000 lb per sq in.

The elongation requirements in Item 8, Table 3, were modified as follows: For structural steel (Column (3)), deduct 1 from the percentage of elonga-

TABLE 3.—CHEMICAL AND PHYSICAL REQUIREMENTS FOR STRUCTURAL STEEL

Item	Description	CARBON STEEL		Silicon steel
		Structural	Rivet	
(1)	(2)	(3)	(4)	(5)
	Chemical Properties:			
1	Carbon (maximum)	....	......	0.40
	Phosphorus (maximum):			
2	Acid process	0.06	0.04	0.06
3	Basic process	0.04	0.04	0.04
4	Sulfur (maximum)	0.05	0.045	0.05
5	Silicon	....	......	0.20 to 0.45
	Physical Properties:			
6	Tensile strength, in pounds per square inch	58 000 to 68 000	52 000 to 60 000	80 000 to 95 000
7	Yield point (minimum), in pounds per square inch	35 000	30 000	45 000
8	Elongation in 8 in. (minimum percentage)	$\frac{1\ 500\ 000}{\text{Tensile strength}}$	$\frac{1\ 500\ 000}{\text{Tensile strength}}$	$\frac{1\ 500\ 000}{\text{Tensile strength}}$
9	Reduction of area (minimum percentage)	42	52	30
	Bend Test*:			
10	Material, $\frac{3}{4}$ in., or less; bend, 180°	Around $D = T$	Flat	Around $D = T$
11	Material more than $\frac{3}{4}$ to $1\frac{1}{4}$ in.; bend, 180°	Around $D = 1.5\ T$	......	Around $D = 1.5\ T$

* D = inside diameter of bend; T = thickness of material.

tion for every $\frac{1}{8}$ in. of thickness greater than $\frac{3}{4}$ in. (minimum elongation, 18%); for silicon steel (Column (5)), deduct 1 from the percentage of elongation for every $\frac{1}{4}$ in. of thickness greater than 1 in. (minimum elongation, 14%).

The area reduction requirements in Item 9, Table 3, were modified as follows: For structural steel deduct 1 from the value in Column (3) for every $\frac{1}{8}$ in. of thickness greater than $\frac{3}{4}$ in. (minimum reduction of area, 35%); for silicon steel, deduct 1 from the value in Column (5) for every $\frac{1}{8}$ in. of thickness greater than $\frac{3}{4}$ in. (minimum reduction of area, 24%).

The average properties of the materials incorporated in the structure, as determined by the standard mill tests, or acceptance tests and ladle analyses, are given in Table 4. In addition to the acceptance tests, check tests were

TABLE 4.—AVERAGE CHEMICAL AND PHYSICAL PROPERTIES OF STRUCTURAL STEEL

	SILICON STEEL		CARBON STEEL	
	Tower	Floor	Tower	Floor
Carbon	0.35	0.35	0.21	0.19
Manganese	0.78	0.76	0.50	0.52
Phosphorus	0.022	0.023	0.018	0.018
Sulfur	0.037	0.034	0.037	0.039
Silicon	0.27	0.27	....	....
Tensile strength, in pounds per square inch	88 800	86 400	63 600	63 400
Yield point, in pounds per square inch	50 800	51 300	38 200	39 400
Percentage elongation, in 8 in.	22	22	28	29
Percentage reduction of area	43	44	52	54
Number of melts	895	257	1 134	484
Number of tests	1 884	669	2 207	971

specified to be made with an extensometer on the usual specimen, $\frac{1}{2}$ in. in diameter. The average results of these tests, as well as the average results of the standard mill tests of the melts from which the check-test specimens

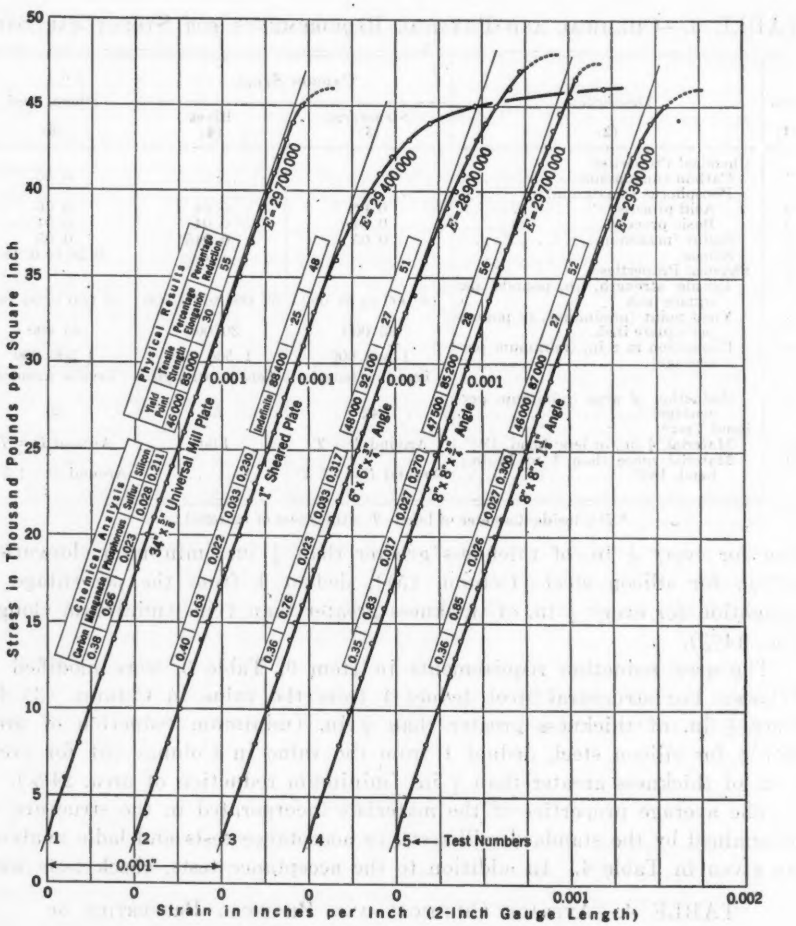


FIG. 2.—STRESS-STRAIN CHARACTERISTICS OF STRUCTURAL SILICON STEEL.

were selected, are given in Table 5. This table shows clearly that the yield point of the material, as measured in the usual manner by the drop of the

TABLE 5.—COMPARISON BETWEEN AVERAGE CHECK TEST AND MILL TEST RESULTS OF STRUCTURAL STEEL

	SILICON STEEL		CARBON STEEL	
	Check tests	Mill tests	Check tests	Mill tests
Tensile strength, in pounds per square inch	88 000	86 900	63 500	64 300
Yield point, in pounds per square inch	47 400	50 700	34 900	39 500
Percentage elongation in 2 in.	28	......	35	....
Percentage elongation in 8 in.	......	22	......	29
Percentage reduction of area	53	41	62	52
Number of melts	146	146	87	87
Number of tests	150	347	87	189

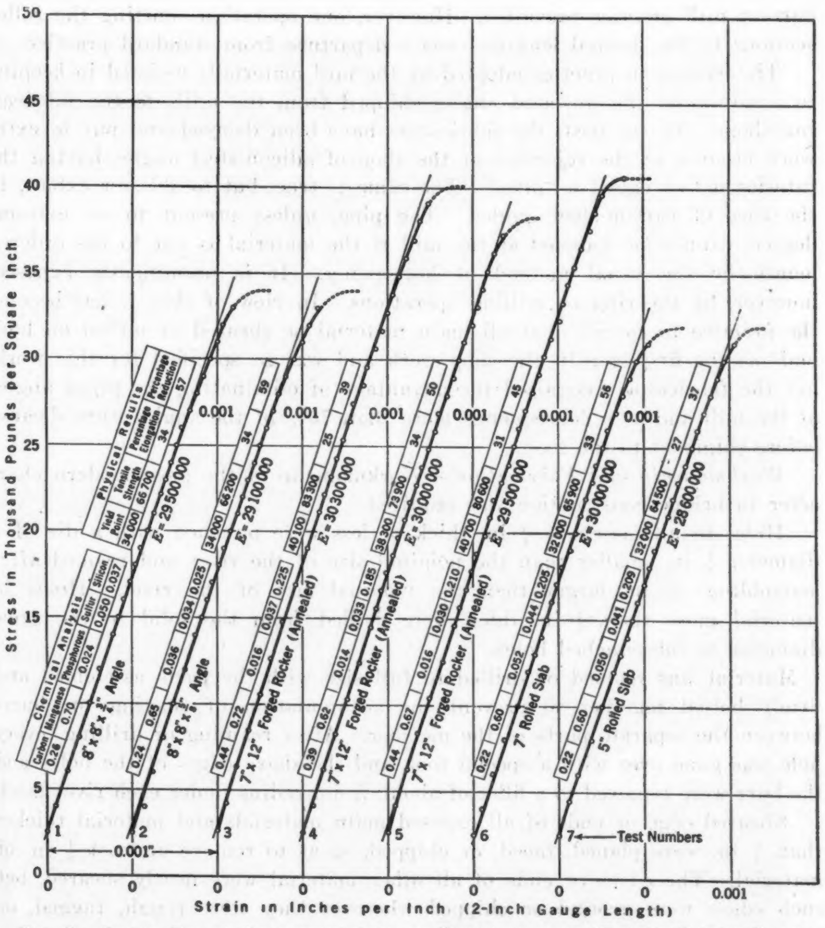


FIG. 3.—STRESS-STRAIN CHARACTERISTICS OF STRUCTURAL AND FORGED CARBON STEEL.

beam of the testing machine in commercial testing of standard mill test specimens, is appreciably higher than the value of this property as measured more accurately on machined specimens otherwise tested. It also shows that with respect to the tensile strength the two methods of testing give similar values. Typical stress-strain diagrams are shown in Figs. 2 and 3. The specimens were machined to a diameter of 0.505 in. Samples for the chemical analyses were milled from tested specimens. The broken-line extension of the curves indicates excessive stretch (not less than 0.009 in. per in.) before the next load increment had been reached. The yield point (the drop of the beam) was considered when plotting the broken lines.

The steel was made by the basic open-hearth process in a number of the larger steel plants in the eastern part of the United States. In general,

current mill practice prevailed. However, one operation—cutting the rolled sections to the desired lengths—was a departure from standard practice.

The change in practice adopted by the mill materially assisted in keeping to a minimum the unsound angles shipped from the mills to the fabricating shops. In the past, the fabricators have been delayed and put to extra work because of the rejection at the shop of silicon-steel angles having the interior defect called a "pipe." The same is true, but to a lesser extent, in the case of carbon-steel angles. The pipe, unless present to an extreme degree, cannot be detected at the mill if the material is cut to the ordered length by the usual method of hot-sawing. It is prominently exposed, however, by shearing or milling operations. In view of this, it has become the practice to specify that all main material be sheared or milled on both ends as the first step in the shop work and was so specified for this work, but the fabricator recognized the advantage of eliminating the piped angles at the mill and as a consequence more than 75% of the angles were sheared before shipment to the shop.

Workmanship and Fabrication.—Workmanship of the most modern character in bridge construction was required.

Holes in all materials $\frac{3}{4}$ in. thick or less were punched with a die of a diameter $\frac{1}{8}$ in. smaller than the nominal size of the rivet and reamed after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivet. Holes in material more than $\frac{3}{4}$ in. thick were drilled from the solid to the same diameter as sub-punched holes.

Material was reamed or drilled to full size with the parts assembled and firmly bolted together to prevent the accumulation of shavings or burrs between the separate parts of the member. After reaming or drilling, every hole was gone over with a special tool, and the sharp edges of the holes and the burr were removed to a fillet of about $\frac{1}{16}$-in. radius under each rivet head.

Sheared edge or ends of all exposed main materials and material thicker than $\frac{3}{4}$ in. were planed, faced, or chipped, so as to remove at least $\frac{1}{8}$ in. of material. The edges or ends of all other material were neatly sheared, but such edges were ground or chipped wherever they were rough, ragged, or irregular.

Rivets were driven by approved pressure tools wherever practicable. The speed and pressure of such tools were regulated to secure the best results in the work. Rivets were driven with pneumatic percussion hammers only when unavoidable and in such cases a pneumatic "bucker-up" was also used wherever possible.

All bearing surfaces were faced to a smooth surface, cut accurately to the proper angle with the axis of the member, so that the abutting members would have full and even bearing when properly aligned. The ends of the sections of the chords and columns were faced after they had been riveted with the exception of the projecting splice-plates. Where necessary, the sections were provided with suitable blocks or plates, spaced near each end to hold the plates and angles firmly in their proper relative position while the ends were being faced.

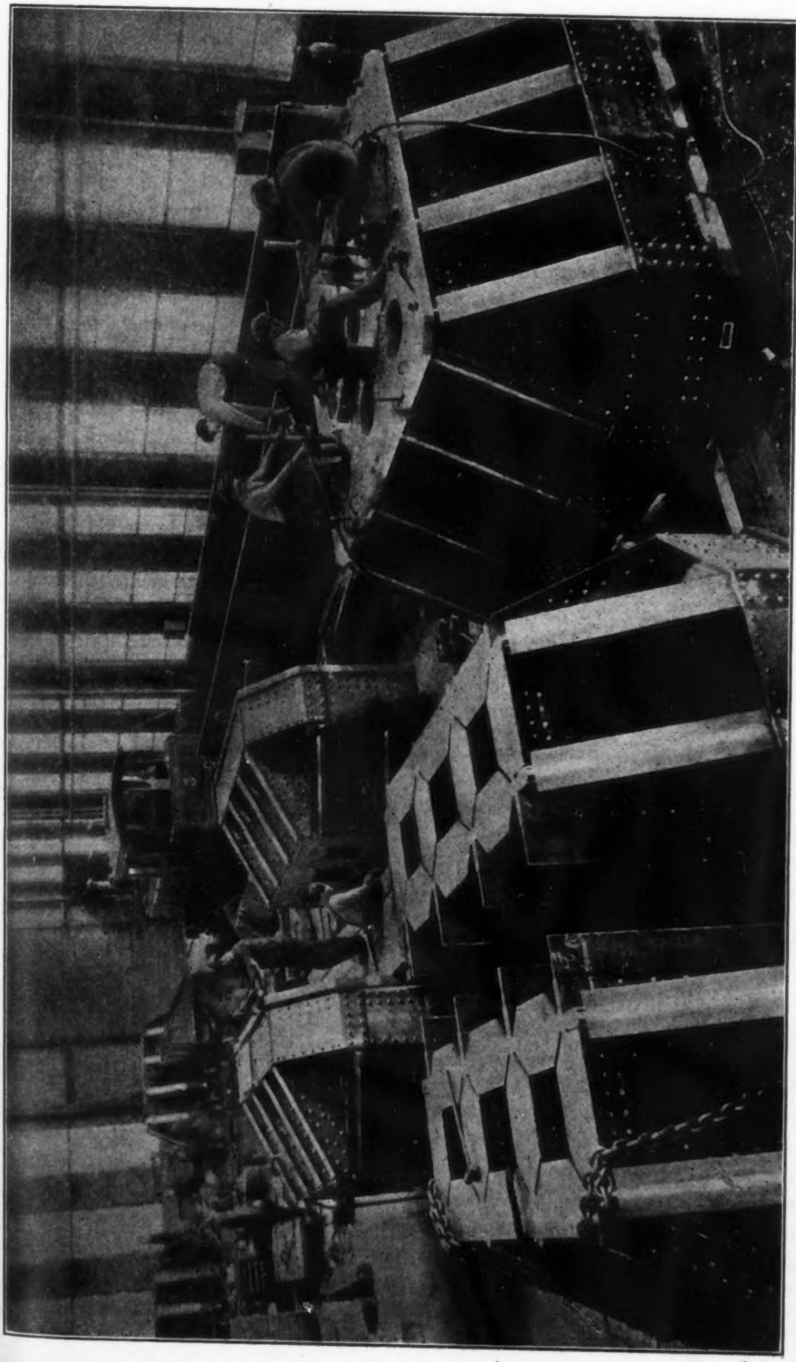


FIG. 4.—TOWER PEDESTALS IN PROCESS OF FABRICATION, GEORGE WASHINGTON BRIDGE.

All field splices in the wind chords of the floor system and, with a few exceptions, the field splices in the columns of the towers were assembled at the shop, then reamed or drilled with the abutting members in close contact and correct alignment, after which the splice material was dis-assembled, the burrs were removed, surfaces cleaned and painted, and the material was re-assembled. All other field connections were reamed or drilled to metal templates with hardened steel bushings set into accurately drilled steel plates.

The structural steel parts of the anchorages and the suspended structure were fabricated by the procedure more or less common for the types of members involved. The tower pedestals and column sections, on the other hand, are unusual in size and shape and, therefore, required exceptional methods of fabrication which are worthy of special mention. Accordingly, an outline is presented of the shop procedure used in the manufacture of these members by the three companies that supplied the materials for the tower proper. The American Bridge Company fabricated the members of the New York tower from the pier to about mid-height; the Bethlehem Steel Company, a like part of the New Jersey tower; and the McClintic-Marshall Company, the remainder.

Pedestals.—The pedestals are made of rolled carbon-steel plates and angles. They were designed with a vertical field splice in order to facilitate shipment, but only those for the New York tower were so fabricated. These members for the New Jersey tower were riveted in the shop to form a unit, differing

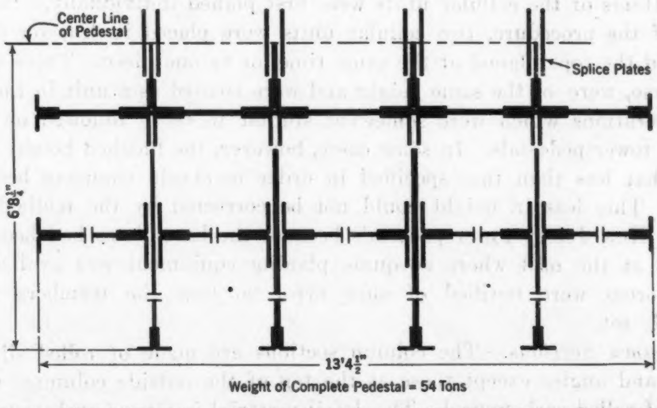


FIG. 5.—CROSS-SECTION OF HALF-PEDESTAL SHOWING FITTING OPERATIONS.

from the other pedestals in one other respect, namely, the cap-plate was made of one piece instead of two.

The pedestals are of cellular construction as shown in Figs. 4 and 5. Before fitting, the top and bottom edges of the web-plates were planed to facilitate the setting of the cap and base angles. In this edge-planing an allowance of about $\frac{5}{16}$ in. in depth of plate was made to permit further planing subsequent to the operation of assembling. The same allowance for planing was made in the sheared length of the stiffener angles.

As previously mentioned, the fabricator of the New Jersey tower pedestals elected to plane and ship each pedestal as a unit. Accordingly, the cellular part of a pedestal was fabricated in two sections, as designed, after which they were fitted together and riveted at the vertical splice, the resulting assembly being handled as a unit through the subsequent operations performed in the machine shop. The top of this assembly was finished first on a boring mill, after which the member was turned upside down and finished on the bottom. The two-piece base-plate, fine finished on the top side and rough finished on the bottom to about $\frac{1}{8}$ in. more than the detailed thickness, was fitted and riveted. The pedestal was then finished to the detailed over-all height by planing the bottom of the base-plate. To complete the member, the one-piece cap-plate, already planed to size and drilled to template, was clamped in place and used as a template for drilling the holes in the cap angles. By this method of fabrication, a pedestal of the specified height was built, in which all stiffeners and web-plates were in full bearing with the cap and base-plates. The slight loss sometimes produced in the height of the cellular unit by excessive planing of any part to secure complete bearing surfaces was corrected merely by increasing the thickness of the base-plate.

The New York tower pedestals were fabricated and shipped as two-piece units to accommodate the shop equipment. The bed of the planer was not wide enough to support a complete pedestal, but it had sufficient length for two half-pedestals. In conducting the planing operations on these members the bottoms of the cellular units were first planed individually. In the next step of the procedure, two cellular units were placed in line on the planer bed and the tops planed at the same time, or as one piece. These two parts, of course, were of the same height and were treated as a unit in the succeeding operations which were somewhat similar to those followed on the New Jersey tower pedestals. In some cases, however, the finished height was made somewhat less than that specified in order to obtain complete bearing surfaces. This loss in height could not be corrected by the method followed on the New Jersey tower pedestals because the base-plates had been finished to size at the mill where adequate planing equipment was available. The field forces were notified of such error so that the members would be properly set.

Column Sections.—The column sections are made of rolled silicon steel plates and angles except those at the top of the outside columns, which are made of rolled carbon steel. The detail material is also of carbon steel except for the gusset-plates of the main bracing, which are of silicon steel. The column sections are one-piece members except some of the upper sections, which were designed with vertical field splices.

In general, the web-plates were sub-punched and the shaft angles were sub-drilled. Most of the material at the splices, both horizontal and vertical, was drilled from the solid while assembled and all other field connections were drilled full size through metal templates after the members had been milled.

Some difficulty was experienced in fitting the heavy gauge angles because the legs were not at a right angle to each other. The mill was not successful

in correcting this condition of the angles. In order to secure close contact of metal to metal—and therefore tight rivets—the parts had to be completely bolted. Illustrations of typical fitting operations are shown in Figs. 6 and 7. The sequence of the principal fitting operations as followed at one of the

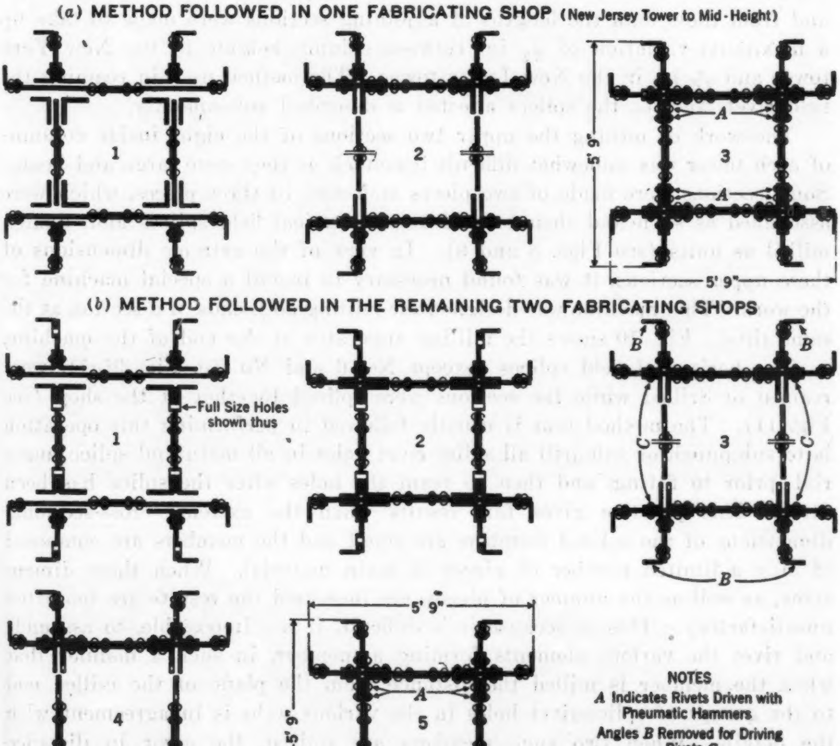


FIG. 6.—SEQUENCE OF PRINCIPAL FITTING OPERATIONS ON TYPICAL TOWER COLUMN SECTIONS.

fabricating shops is demonstrated by the three steps in Fig. 6(*a*). The remaining two fabricating shops followed the sequence illustrated by the five steps in Fig. 6(*b*).

In conducting milling operations on a column section careful consideration was given to the finished length checked by steel measuring bars. Any variation between the specified and measured lengths was included in the adjoining section. However, the need for correcting possible accumulation of errors from any cause to procure true and level seats for the grillages supporting the cable saddles was anticipated. Accordingly, the specifications

required that the top sections of the columns supporting the saddles directly, as well as the top sections of the remaining columns at the same level, be milled at the shop to lengths (determined by a field survey) that would bring the column tops of each tower to a plane. The field checks were made when the towers had been erected to the horizontal field splice, No. 10 (see Fig. 9(*a*)), and from these data the lengths of adjoining sections were made to take up a maximum variation of $\frac{7}{32}$ in. between column heights in the New York tower and $\frac{3}{16}$ in. in the New Jersey tower. The method used in reaming the field rivet holes of the splices affected is described subsequently.

The work of milling the upper two sections of the eight inside columns of each tower was somewhat difficult inasmuch as they were large and heavy. Some sections were made of two pieces and some of three pieces, which were assembled as such and then reamed at the vertical field splices and, finally, milled as units (see Figs. 8 and 9). In view of the extreme dimensions of these upper sections, it was found necessary to install a special machine for the work. This machine was designed for milling both ends of a section at the same time. Fig. 10 shows the milling apparatus at one end of the machine.

The horizontal field splices (except No. 6 and No. 10, Fig. 9(*a*)), were reamed or drilled while the sections were spliced together at the shop (see Fig. 11). The method that is usually followed in performing this operation is to sub-punch or sub-drill all splice-rivet holes in all main and splice material prior to fitting, and then to ream the holes after the splice has been fitted. This practice gives fair results when the extreme cross-sectional dimensions of the spliced members are small and the members are composed of only a limited number of pieces of main material. When these dimensions, as well as the number of pieces, are increased the results are too often unsatisfactory. This is because it is difficult, if not impossible, to assemble and rivet the various elements forming a member, in such a manner that when the member is milled the distance from the plane of the milled end to the groups of splice-rivet holes in the various webs is in agreement with the details. When two such members are spliced, the error in distance between the set of holes in the splice in one member and those in the other is quite often increased. Consequently, the splice material which previously had been sub-punched or sub-drilled according to detail would not fit these conditions, and imperfect holes would result when they are reamed to full size.

Two methods were used to obtain good holes in the horizontal splices of the columns. At two plants the following practice was adopted: A few holes in the main material and corresponding holes in the splice material on the inside of the member were sub-punched or sub-drilled to permit assembling them. All holes were sub-punched in that splice material which was on the outside of the member and which served for use as templates in drilling the full-sized holes. At the other plant all holes were sub-punched in the splice-plates and sub-drilled to metal templates in the main material and splice angles. However, the holes in the main material were not sub-drilled until after the members had been milled, so that advantage could

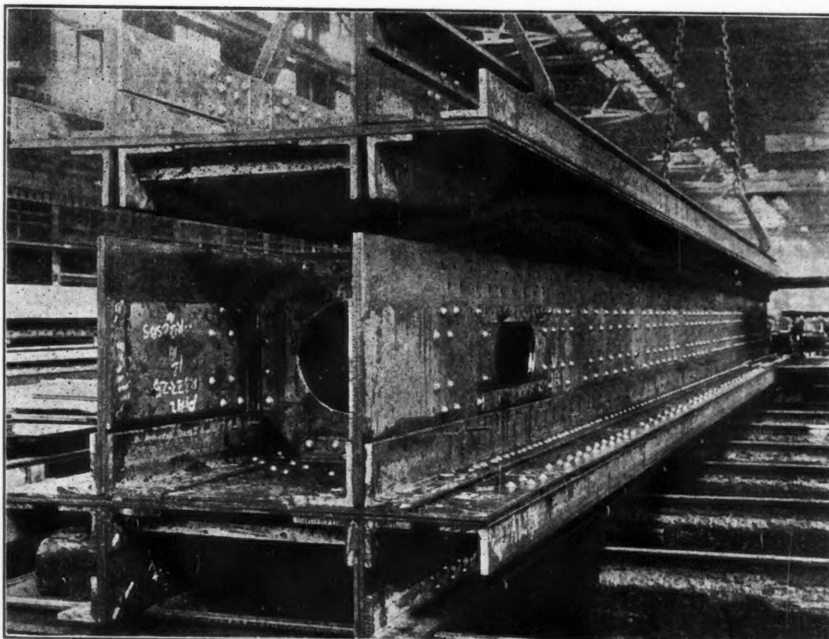


FIG. 7.—FINAL FITTING OPERATION ON TYPICAL TOWER COLUMN SECTION.

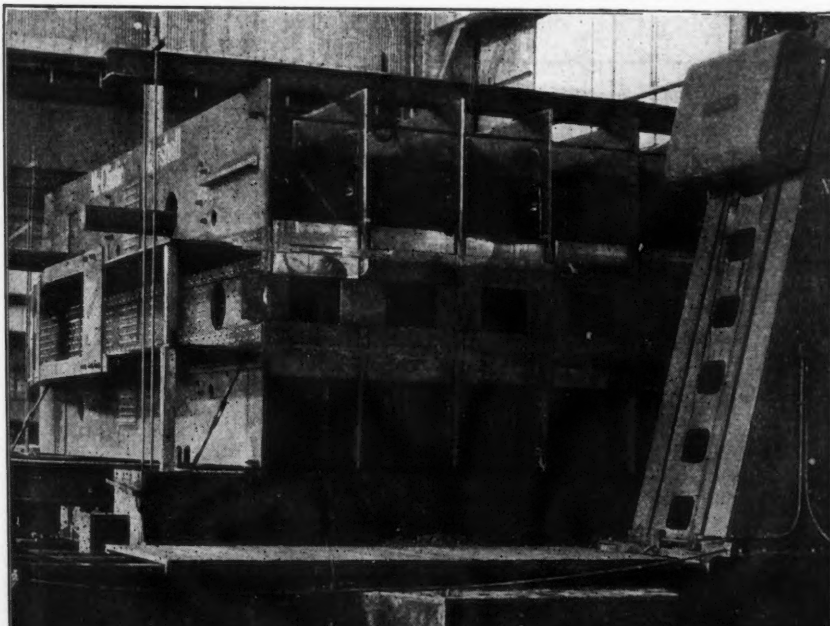


FIG. 8.—MILLING TOP SECTION OF TOWER COLUMN.

Fig. 1—[illegible]

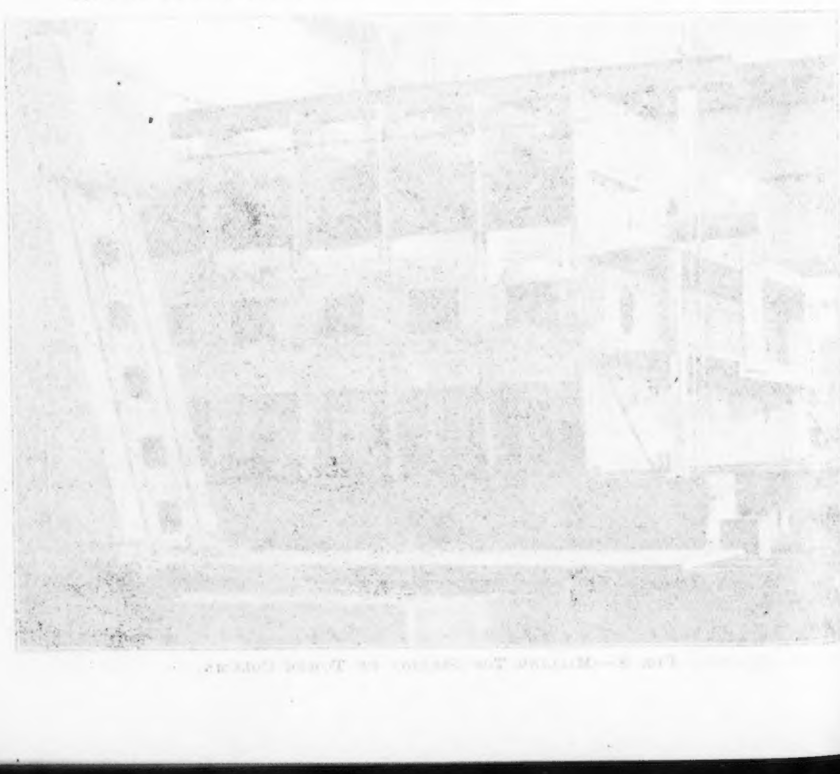


Fig. 2—Milling The Facing of Tower Columns.

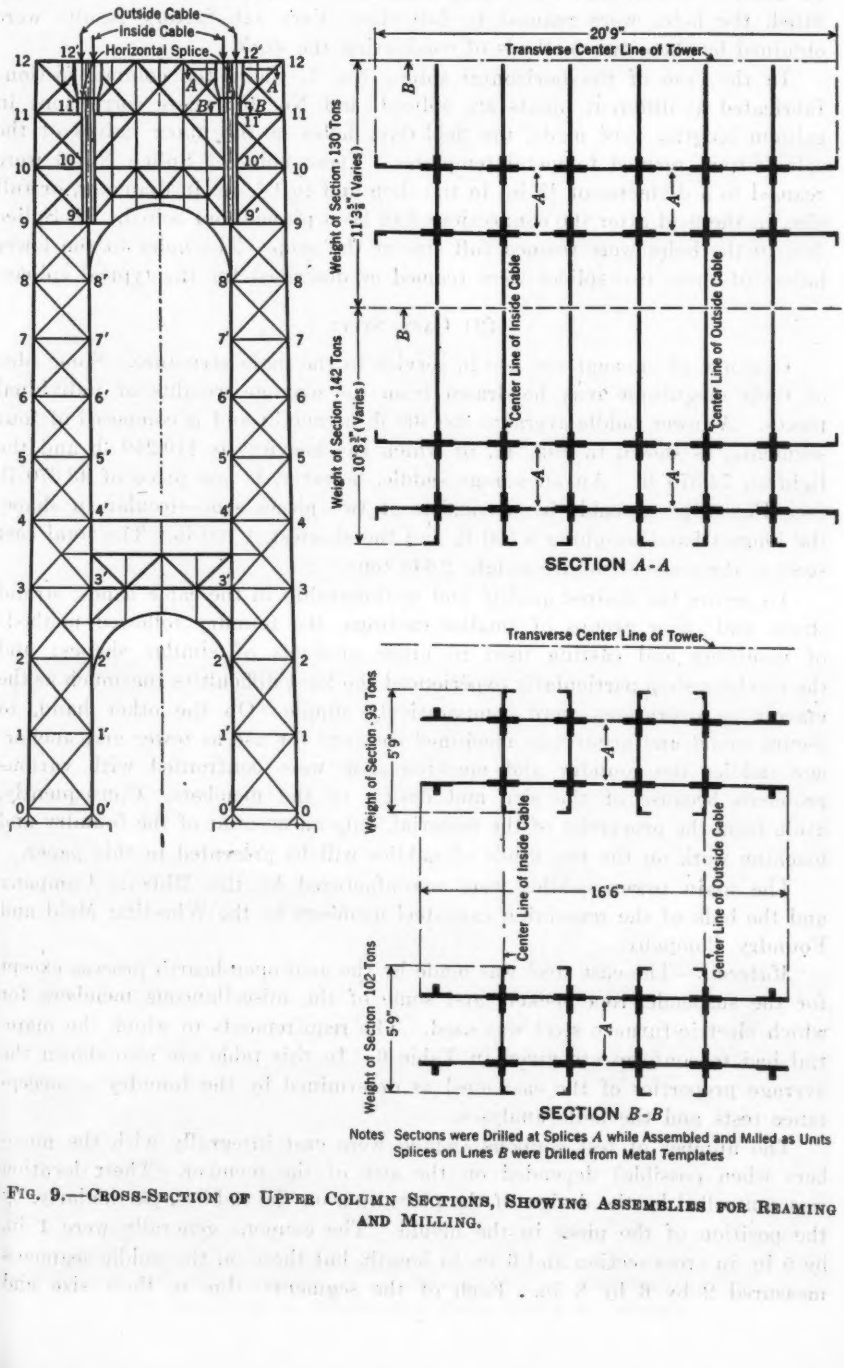


FIG. 9.—CROSS-SECTION OF UPPER COLUMN SECTIONS, SHOWING ASSEMBLIES FOR REAMING AND MILLING.

be taken of milled ends for setting templates. After the splice was properly fitted, the holes were reamed to full size. Very satisfactory results were obtained by these two methods of conducting the work.

In the case of the horizontal splice, No. 6, where the column sections fabricated at different plants are spliced, and No. 10, where corrections in column lengths were made, the field-rivet holes in the upper halves of the splices were reamed to metal templates. These holes of Splice No. 6 were reamed to a diameter of $\frac{15}{16}$ in. in the shop and to $1\frac{1}{16}$ in. in diameter, or full size, in the field after the connections had been pinned and bolted. In Splice No. 10 the holes were reamed full size at the shop. The holes in the lower halves of these two splices were reamed as described for the typical splices.

(3) Cast Steel

Castings of unusual size are in service in the main structure. Some idea of their magnitude may be drawn from the average weights of individual pieces. A tower saddle averages 359 400 lb in weight and is composed of four segments, as shown in Fig. 12, of which the heaviest is 110 240 lb and the lightest, 74 370 lb. An anchorage saddle, however, is one piece of 43 670 lb (see Fig. 13). A cable band consists of two pieces semi-circular in shape, the longest band weighing 6 500 lb and the shortest, 1 150 lb. The total cast steel in the main structure weighs 2 643 tons.

To secure the desired quality and workmanship in the cable bands, strand shoes, and other groups of smaller castings, the foundry followed methods of moulding and casting used in other products of similar shapes; and the machine shop particularly experienced the least difficulties inasmuch as the machining operations were comparatively simple. On the other hand, to secure sound and accurately machined castings for use as tower and anchorage saddles the foundry and machine shop were confronted with various problems because of the size and design of the members. Consequently, aside from the properties of the material, only an account of the foundry and machine work on the two kinds of saddles will be presented in this paper.

The main tower saddles were manufactured by the Midvale Company and the bulk of the remaining cast-steel members by the Wheeling Mold and Foundry Company.

Material.—The cast steel was made by the acid open-hearth process except for the suspender-rope sockets and some of the miscellaneous members for which electric-furnace steel was used. The requirements to which the material had to conform are given in Table 6. In this table are also shown the average properties of the cast steel as determined by the foundry or acceptance tests and the ladle analyses.

The number of test coupons (which were cast integrally with the members when possible) depended on the size of the member. Their location was controlled by the design of the piece and, on the saddles, particularly, by the position of the piece in the mould. The coupons generally were 1 in. by 5 in. in cross-section and 6 in. in length, but those on the saddle segments measured 2 by 6 by 8 in. . Each of the segments—due to their size and

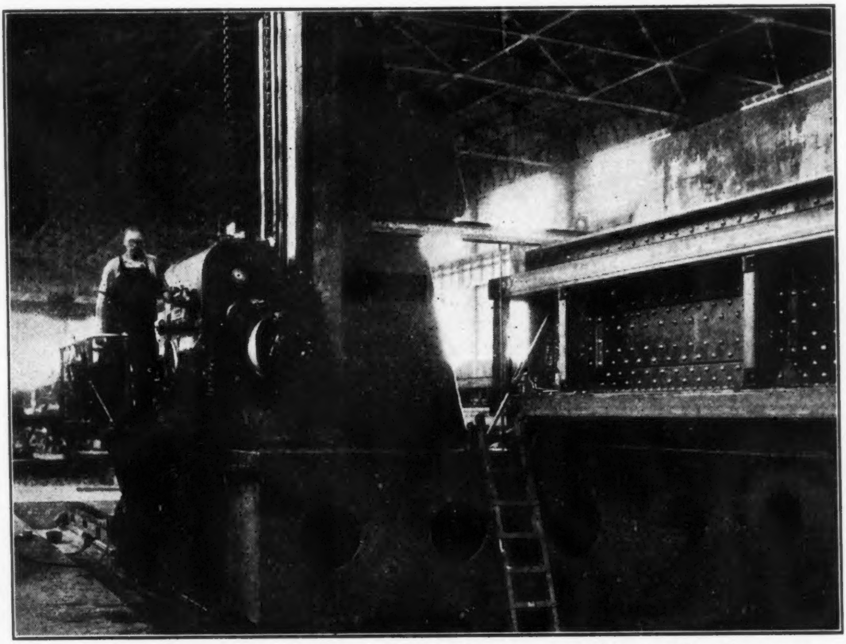


Fig. 10.—View Showing One End of Large Machine for Milling Top Sections of Tower Columns.

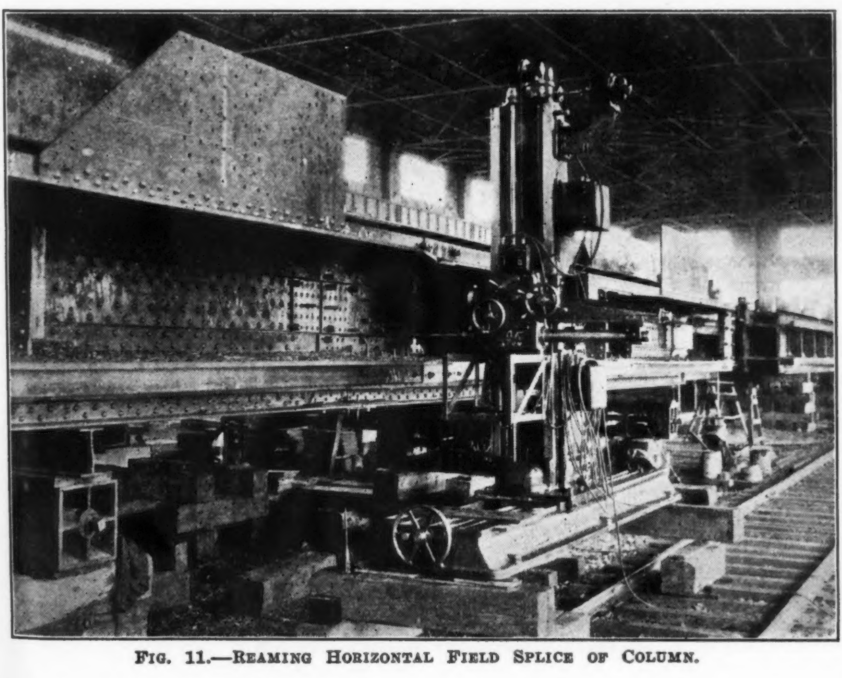


Fig. 11.—Reaming Horizontal Field Splice of Column.

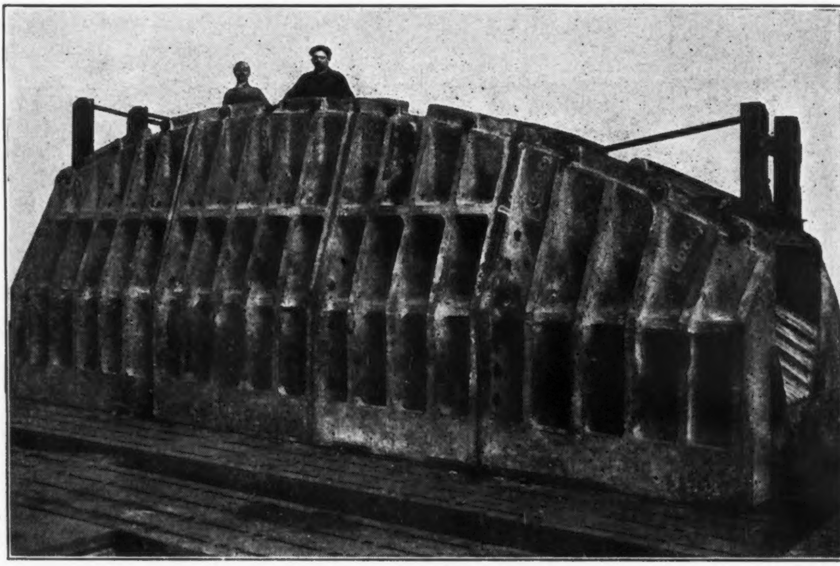


FIG. 12.—VIEW OF MAIN TOWER SADDLE.

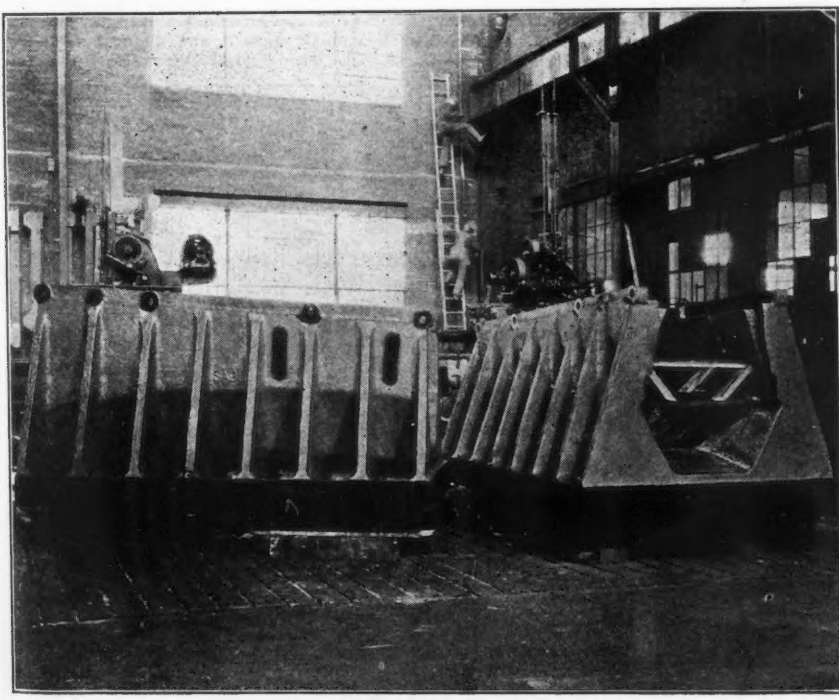


FIG. 13.—VIEW OF ANCHORAGE SADDLES.

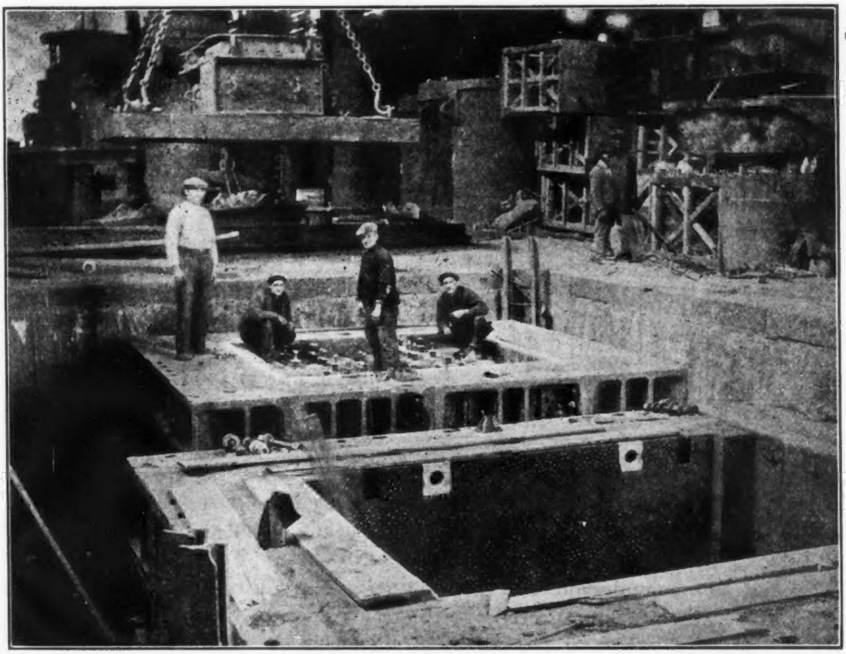


FIG. 14.—MOULDS FOR TOWER SADDLE SEGMENTS. IN FOREGROUND, MOULD READY TO RECEIVE CORES; IN BACKGROUND, CLOSING MOULD; "COPE COVER" IN CRANE HOOKS.

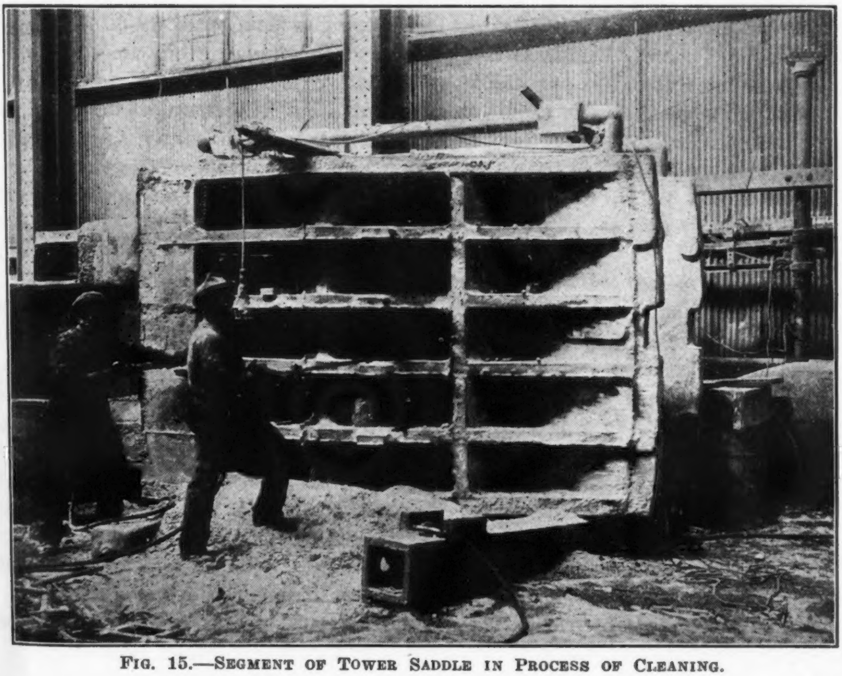


FIG. 15.—SEGMENT OF TOWER SADDLE IN PROCESS OF CLEANING.

TABLE 6—SPECIFIED AND AVERAGE PROPERTIES OF CAST STEEL

Part of structure	Number of tests	PHYSICAL				CHEMICAL				
		Yield point, in pounds per square inch	Tensile strength, in pounds per square inch	Percentage elongation in 2 inches	Percentage reduction of area	Carbon	Manganese	Phosphorus	Sulfur	Silicon
Specified requirements		35 000 min.	65 000 min.	20 min.	30 min.	....	....	0.06 max.	0.05 max.	....
Main tower saddles...	64	39 000	72 300	26	41	0.31	0.61	0.034	0.039	0.34
Anchorage saddles....	16	39 500	75 200	27	41	0.34	0.65	0.044	0.032	0.34
Cable bands..........	659	37 800	71 000	30	49	0.29	0.69	0.040	0.029	0.33
Strand shoes..........	510	38 800	75 500	27	41	0.30	0.73	0.039	0.032	0.34
Suspender rope sockets *..............	85	41 600	69 700	28	43	0.25	0.63	0.036	0.040	0.29
Miscellaneous †.......	64	43 200	76 300	26	39	0.25	0.69	0.034	0.044	0.40
Average.........	..	38 700	72 900	29	45	0.28	0.67	0.038	0.035	0.34

* Electric furnace steel. † Includes both acid open-hearth and electric-furnace steel.

the fact that they were cast with the base horizontal and in the cope, or the top of the mould — had eight coupons spaced at intervals on both ribbed sides, from which two were selected to provide two tension and two bend test specimens. Affording a similar choice, each anchorage saddle had one coupon in the vicinity of each corner of the base, these members being cast with the base vertical. A half-cable band carried two coupons attached to the bore side. Weight governed the number of coupons on bands as well as on other castings, thus providing one tension and one bend test for each casting of 500 lb, or more, and sufficient specimen material for two tests of each kind per melt and annealing charge for lighter castings.

Foundry Work.—The two foundries in which the different saddles were made followed practically the same pouring, cleaning, and annealing procedure. The molten metal was introduced at the lowest point of the mould; the castings were cleaned by means of pneumatic chipping hammers and sand-blasting; and the annealing operation was conducted, of course, with the view of obtaining uniform structure throughout the castings in the charge.

The methods of moulding the two kinds of saddles, however, differed widely. The tower saddle segments were moulded in sectional flasks with the aid of patterns and dry sand cores. (See Fig. 14.) Previous to the operation of setting the cores the mould was dried by coke and coal fires lowered into place. The segments were cast upside down in respect to their position in the structure so that, with the introduction of the molten metal at the lowest point of the mould, the dirt and slag tended to float to the base of the segment where removal of contaminated metal was most practical. Therefore, an allowance was made in moulding to provide 1 in. of excess metal on the base, but this was increased subsequently to 3 in. to insure elimination of all unsound metal by machining.

Fig. 15 shows a segment of a tower saddle in process of cleaning. The segment is resting on one end with the cable groove to the right and the

base to the left. The gates, or the points where the metal entered the mould, are clearly indicated, being the points on either side of the cable groove where the runners are attached to the casting. The two risers, provided to "feed" the casting during solidification, are shown attached to the base; the test coupons, attached to the ribs, are clearly in evidence.

In the case of the anchorage saddles, the mould was made in a pit by a method known as the "core assembly". The procedure consisted of forming a mould of dry sand cores and of ramming sand between the cores and the sides of the pit. The members were cast on end; in other words, with the base in a vertical position.

Machine Work.—The machine work on an anchorage saddle was confined chiefly to simple planing operations on the base surface and flanges, and to a dressing process on the cable groove. The latter, a grinding operation, removed the high spots the locations of which were determined by a metal template.

The work of machining a tower saddle was a more involved operation because of the intricate cable groove and the number of segments After milling the base of each segment with an allowance in thickness to permit further planing, the ends were planed to secure tight joints in the completed saddle. Because of planing-machine limits, only three segments of the four were bolted together and finished on the base at one time. To complete the planing work on the base, one section was removed and the fourth one added to the other end, the base of the latter being finished to the plane of the other two. Finally, after the four segments had been assembled into a saddle supported on its side, the cable groove was machined. (See Fig. 16.) During this operation the member remained stationary while two milling machines traveled on the same curved track to shape the cable groove. The finishing cuts were made by one machine only in order to obviate the ridge that might result from the slight variation in the cutting action of the two machines.

(4) Cable Wire

The four cables are composed of cold-drawn galvanized steel wire about 0.196 in. in diameter. They were constructed from long lengths of wire made continuous by means of splices. One such length as shipped from the factory to the bridge site would measure about 28 miles and would weigh approximately $7\frac{1}{2}$ tons. The total quantity of wire used is 28 308 tons.

In view of the large tonnage involved (which is about four times that of any previous single order for this kind of material) and also the uniformity of the product, an account of the manufacture in some detail is warranted. All manufacturing operations from the melting of the steel to, and including, the reeling of the finished product were carried out at the same plant. In the principal operations the methods were similar, in general, to the practice found in the cable-wire industry.

This material was manufactured by the John A. Roebling's Sons Company in its plant at Roebling, N. J.

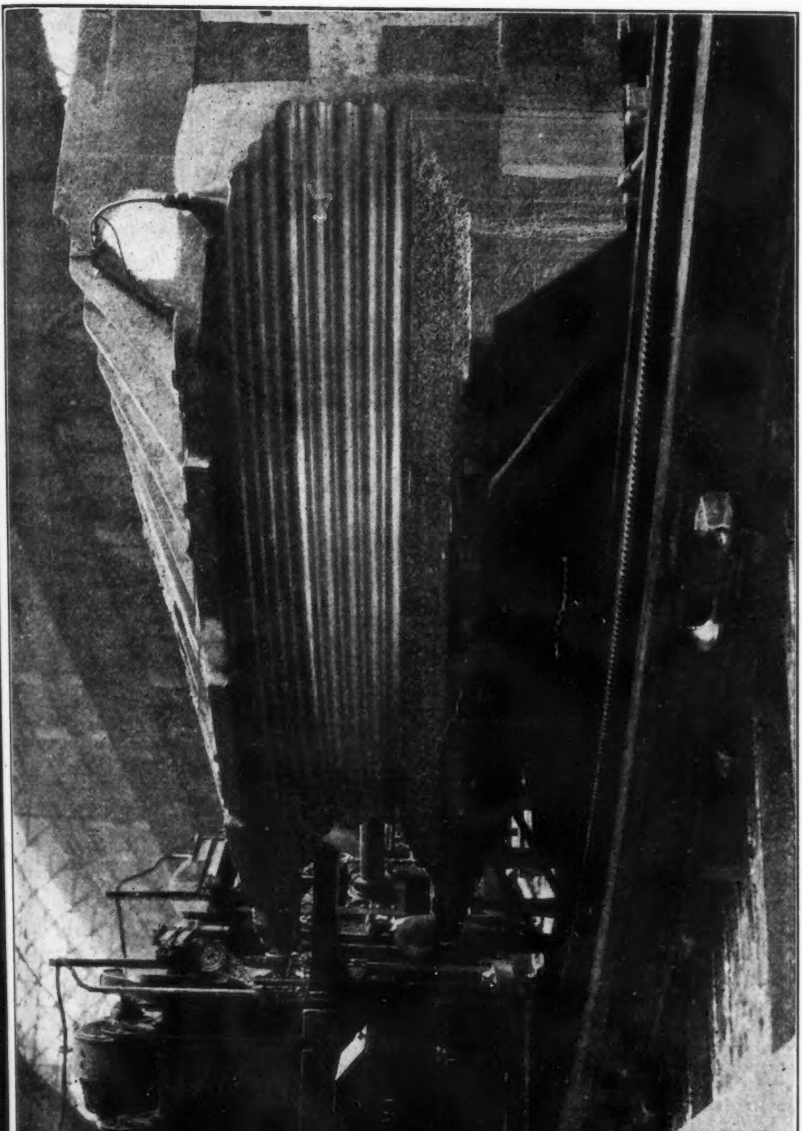


FIG. 16.—MACHINING CABLE GROOVE OF MAIN TOWER SADDLE.

Chemical Properties.—The specifications permitted a maximum carbon content of 0.85% and a phosphorus and a sulfur content of 0.04% each, on ladle analysis. On check analysis of the finished or semi-finished product, an excess beyond these limits was permitted, as follows: 10% in the case of carbon; 25% in the case of phosphorus; and 25% in the case of sulfur. The amounts of the other elements were not indicated.

For determining the percentages of carbon, manganese, phosphorus, sulfur, and silicon in the steel, a ladle analysis and two check analyses were made on samples from each melt. The sample for the ladle analysis was drilled from a test ingot which was cast after about one-half the melt had been teemed or run into the ingot moulds. The two samples for the check analyses were drilled from different billets. One sample was always taken from the top billet, or the billet adjacent to the top discard of the ingot, where the greatest segregation would most likely be found, and the other usually from the bottom billet of another ingot.

The average, maximum, and minimum results of the ladle and check analyses of the 958 melts of steel from which the wire in the cables was manufactured, are given in Table 7.

The degree of uniformity of the steel throughout the melt may be deduced from the amount of each element present as disclosed by the two check

TABLE 7.—CHEMICAL ANALYSES OF CABLE WIRE

	Ladle					Check				
	Carbon	Mangenese	Phosphorus	Sulfur	Silicon	Carbon	Mangenese	Phosphorus	Sulfur	Silicon
Average	0.80	0.63	0.029	0.036	0.19	0.81	0.63	0.029	0.034	0.19
Maximum	0.85	0.83	0.040	0.045	0.32	0.93	0.77	0.042	0.046	0.34
Minimum	0.76	0.50	0.020	0.025	0.11	0.72	0.44	0.021	0.022	0.07

analyses, the samples for which, as previously indicated, were selected with the view of proving the material in this respect. Of the melts showing the greatest variations in amount of each of the three elements considered by the specifications, fifteen showed a difference in carbon content ranging between 0.07 and 0.14%; fifteen, in the case of phosphorus content, between 0.003 and 0.015%; and a like number, in the case of sulfur content, between 0.006 and 0.019 per cent.

Manufacture.—The manufacturing process is a standard one for producing this kind of wire. The materials are combined and melted in open-hearth furnaces. The ingots are rolled into billets which, in turn, undergo further rolling into rods. The rods are patented, pickled and cleaned, coated and baked with drawing vehicle, and, finally, cold-drawn into wire. The wire then passes into the galvanizing unit where it is annealed, cleaned, galvanized, and wound on drums from which it is taken to be reeled for shipment.

The initial operations vary with the manufacturer in some of the details. For this wire, the steel was made in acid open-hearth furnaces in small quantities of about 35 tons, or about 40% less in amount than is the

practice in the ordinary steel plant where the quantity per melt is approximately 50 tons and more. The ingots were cast in hot top moulds to lessen and confine the pipe; they were 14 by 14 in. in cross-section and weighed about a ton. The billets were 4 by 4 in. in section, but, later, were rolled to 2 by 2 in. when additional equipment was placed in operation; their weight in both sizes averaged about 380 lb. The rods rolled from these billets were reduced from a nominal diameter of 0.360 in. to one of 0.192 in. by cold-drawing them through four dies progressively smaller in diameter.

Tests and Physical Properties.—The material was subjected to physical tests before it entered the galvanizing unit and after it had been galvanized. A bend test only was performed on the wire in the first-mentioned or "green wire" stage. The Preece or copper sulfate dip test for thickness of zinc coat, a bend test, and a tensile test determined the quality of the galvanized product. The Preece test was made on 5% of the coils, and all specimens passed it. In addition to these routine tests, investigations were conducted to determine the modulus of elasticity of the wire and to ascertain the degree of uniformity in strength throughout the coil.

Bend tests were made on specimens from one end of every coil or length of "green wire" and from both ends of 10% of the galvanized coils. The test consisted of coiling the specimen for one complete turn on a mandrel $1\frac{1}{2}$ times the nominal diameter of the wire. No failures occurred in the "green wire" coils, and the number of coils which failed after galvanizing was negligible.

The specifications for the tensile test of the galvanized material required determinations of the yield point, tensile strength, and total elongation on both ends of 10% of the coils, but only of the latter two characteristics on both ends of other coils. The minimum values, respectively, for these properties were set at 150 000 and 220 000 lb per sq in. and 4% in 10 in. Furthermore, twelve consecutive tests had to show a minimum average yield point of 153 300 and a minimum average tensile strength of 225 000 lb per sq in. The former was defined as the stress that produced a stretch of 0.7% in 10 in. The cross-sectional area upon which the unit stresses are based, is derived from the gross diameter which includes the galvanizing.

The tensile tests presented a minor problem in load application which had to be solved at the beginning, because of the curve in the specimen to which the extensometer had to be attached. The specimens were curved to a radius of about 3 or 4 ft, or to a radius somewhat greater than that of the drums on which the wire had been wound. Obviously, such a condition affects the degree of accuracy with which the yield point may be measured. Experiments disclosed the fact that under a load of about 650 lb, and with a distance of 24 in. between the jaws of the testing machine, the curve is practically eliminated; that is, the 12-in. central portion of the specimen appeared to be straight as observed by the aid of a straight-edge. Furthermore, other preliminary tests furnished a value for the modulus of elasticity of 28 000 000 lb per sq in. for the material. Based on these data, a procedure was established for the routine yield-point tests in which an initial load of 850 lb was applied to the specimen before the attachment of the two

extensometers. The initial load of 850 lb was calculated to produce, for all practical purposes, an elongation of 0.1% for gross diameters within the specified limits of 0.192 in. and 0.200 in. The calculated percentage of stretch in these tests is more nearly correct when the diameter of the specimen is 0.196 in. which happens to be the average diameter of the wire actually incorporated in the cables. In the other tests, where only the tensile strength and total elongation were desired, an initial load of about 2 000 lb was applied before attaching one extensometer, but no addition was made to the measured stretch because the latter, in all cases, was well above the specified minimum total elongation.

The use of two extensometers in connection with the yield-point test was necessary for the reason that a sensitive instrument required to indicate small elongations would be injured from blows caused by the rupture of the specimen. For measuring the stretch from the initial load to the yield point a 10-in. extensometer was used and then removed. This extensometer was equipped with an Ames dial registering increments of 0.001 in. of stretch. The other or sturdier instrument, which was attached to the specimen at the same time, had a dial graduated to hundredths of an inch, and this instrument remained on the specimen until rupture, so that the operator could observe the indicator at the instant of rupture.

The average results of 26 274 tests, or those for which the yield point was measured, are given in Table 8. They cover 958 melts of wire.

TABLE 8.—Results of Yield-Point Tests of Cable Wire

	Minimum	Average	Maximum
Tensile strength, in pounds per square inch	220 000	234 000	259 000
Yield point, in pounds per square inch	150 000	184 000	202 000
Percentage elongation, in 10 in.	....	6.0	....
Diameter, in inches	....	0.1965	....

Rejections amounted to 0.7% of the total wire used. Of this quantity, 74% represents material having physical and chemical properties outside the specified limits; 13%, surface inperfections; and the remainder includes material which showed such undesirable qualities as "off gauge", kinks or waves, and brittleness.

The modulus of elasticity of the wire was determined for use in connection with the study of testing apparatus and procedure and for other purposes, outside the scope of this paper. The modulus tests were made with 300-in. gauge lengths on specimens, each from a different melt. In the preliminary investigations, eight tests were made, the results of which indicated an average modulus of 28 000 000 lb per sq in. Additional tests, ten in number, made during the manufacturing period, revealed an average value of 28 700 000 lbs per sq in. Although the results of the latter tests would seem to indicate that this wire has a modulus of elasticity somewhat higher than was assumed when establishing the routine testing procedure used in performing the yield-point test, no change was made in the procedure because the calculated stretch for the initial load remained the same for all practical

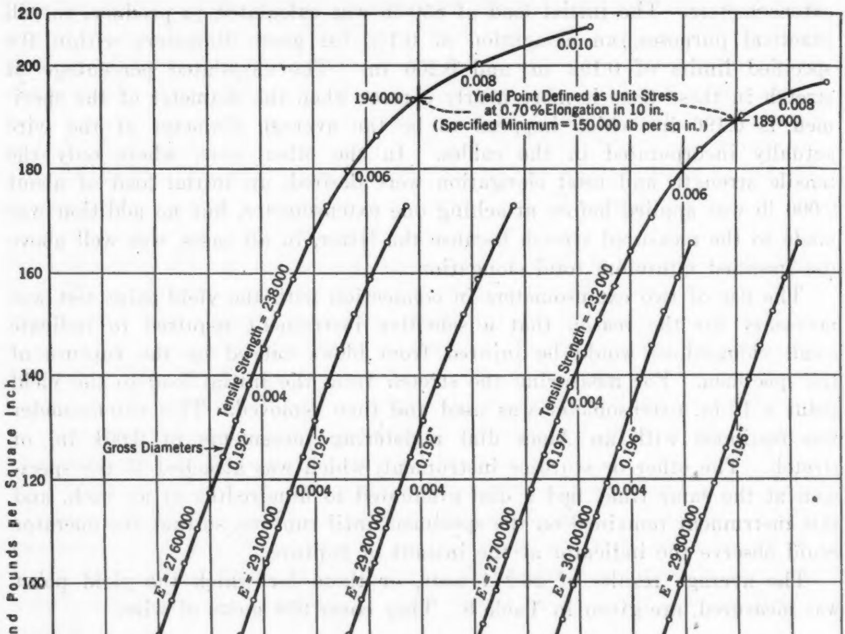


FIG. 17.—STRESS-STRAIN CHARACTERISTICS OF CABLE WIRE.

purposes. The results of some of the tests have been plotted in Fig. 17. The gross diameters indicated on each curve include the thickness of the galvanizing. The tests in Group A (A_1, A_2, and A_3) were made on one length of wire. Those in Group B (B_1, B_2, and B_3) likewise were made on one length of wire, but from a different melt. The test results indicated by

TABLE 9.—PHYSICAL PROPERTIES THROUGHOUT COILS OF CABLE WIRE
(Specimens Taken Every 100 Feet)

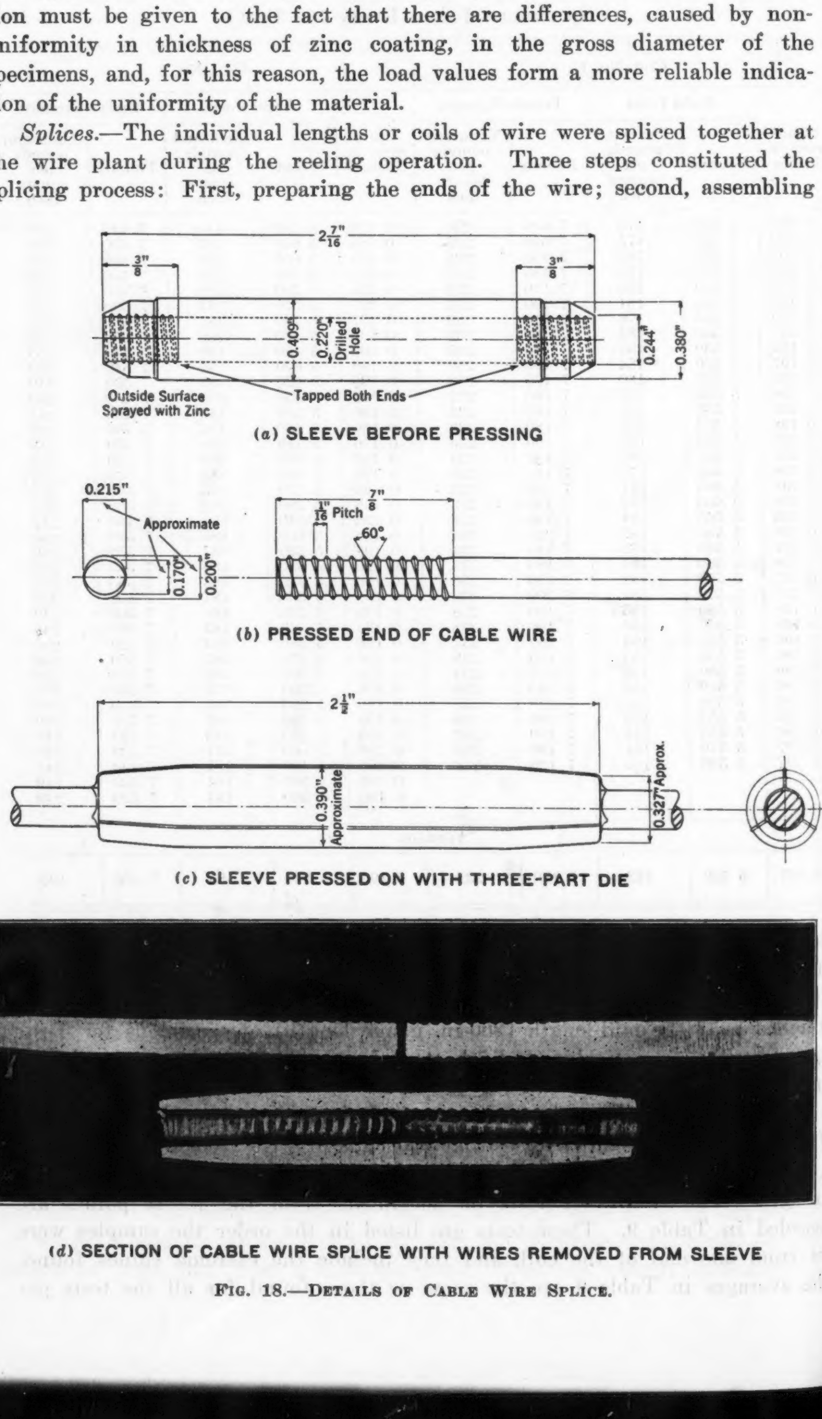
COIL No. 1					COIL No. 2				
	Yield Point		Tensile Strength			Yield Point		Tensile Strength	
Diameter, in inches	Pounds	Thousand pounds per square inch	Pounds	Thousand pounds per square inch	Diameter, in inches	Pounds	Thousand pounds per square inch	Pounds	Thousand pounds per square inch
0.197	5 430	178	7 020	230	0.196	5 580	185	7 440	247
0.200	5 490	175	7 060	225	0.196	5 640	187	7 470	248
0.197	5 560	182	7 180	236	0.196	5 600	186	7 500	249
0.198	5 520	179	7 180	233	0.197	5 180	170	7 550	248
0.198	5 540	180	7 170	233	0.196	5 700	189	7 450	247
0.198	5 580	181	7 190	234	0.197	5 620	184	7 530	247
0.199	5 460	176	7 210	232	0.197	5 700	187	7 530	247
0.197	5 550	182	7 160	235	0.196	5 700	189	7 560	251
0.197	5 530	181	7 180	236	0.196	5 420	180	7 540	250
0.197	5 620	184	7 190	236	0.196	5 600	186	7 540	250
0.198	5 530	180	7 170	233	0.196	5 520	183	7 520	249
0.198	5 590	182	7 160	233	0.196	5 550	184	7 530	250
0.197	5 550	182	7 160	235	0.196	5 530	183	7 450	247
0.198	5 560	181	7 180	233	0.196	5 540	184	7 490	248
0.197	5 560	182	7 190	236	0.196	5 410	179	7 470	248
0.197	5 560	182	7 180	236	0.196	5 060	168	7 570	251
0.198	5 500	179	7 160	233	0.196	5 560	184	7 600	252
0.198	5 580	181	7 180	233	0.196	5 570	185	7 540	250
0.198	5 600	182	7 180	233	0.196	5 500	182	7 560	251
0.196	5 560	184	7 190	238	0.196	5 540	184	7 450	247
0.198	5 590	182	7 220	234	0.196	5 600	186	7 540	250
0.197	5 540	182	7 140	234	0.196	5 630	187	7 550	250
0.198	5 560	181	7 180	233	0.196	5 590	185	7 540	250
0.197	5 550	182	7 180	236	0.196	5 620	186	7 550	250
0.197	5 520	181	7 200	236	0.196	5 560	184	7 500	249
0.196	5 580	185	7 190	238	0.196	5 560	184	7 500	249
0.196	5 540	184	7 180	238	0.196	5 520	183	7 500	249
0.196	5 540	184	7 160	237	0.196	5 580	185	7 520	249
0.196	5 530	183	7 190	238	0.196	5 570	185	7 520	249
0.196	5 580	185	7 210	239	0.196	5 560	184	7 480	248
0.196	5 610	186	7 240	240	0.196	5 260	174	7 560	251
0.196	5 530	183	7 220	239	0.196	5 680	188	7 520	249
0.197	5 560	182	7 200	236	0.196	5 580	185	7 520	249
0.196	5 550	184	7 160	237	0.196	5 580	185	7 520	249
0.197	5 510	181	7 190	236	0.196	5 160	171	7 520	249
0.197	5 550	182	7 120	234	0.196	5 460	181	7 490	248
					0.196	5 560	184	7 500	249
					0.196	5 540	184	7 520	249
AVERAGE									
0.197	5 550	182	7 170	235	0.196	5 540	183	7 520	249

Curves A_1 and B_1 were obtained with acceptance test apparatus on specimens (10-in. gauge length) from respective lengths. The test results indicated by Curves A_2 and B_2, were obtained with a 10-in. acceptance-test extensometer attached near the mid-length (300-in. gauge length) of specimens for Tests A_3 and B_3, respectively, and the two sets of tests were conducted simultaneously.

The degree of uniformity in the physical properties throughout the length of a coil was investigated by testing two coils of wire from different melts. A specimen was taken every 10 ft and tested for yield point and tensile strength. The results of tests on specimens from the 100-ft points are recorded in Table 9. These tests are listed in the order the samples were cut from one end of the coil, and they include the extreme values found. The averages in Table 9 are the same as those found for all the tests per

coil. It should be noted that in studying the values in this table consideration must be given to the fact that there are differences, caused by non-uniformity in thickness of zinc coating, in the gross diameter of the specimens, and, for this reason, the load values form a more reliable indication of the uniformity of the material.

Splices.—The individual lengths or coils of wire were spliced together at the wire plant during the reeling operation. Three steps constituted the splicing process: First, preparing the ends of the wire; second, assembling



(*a*) SLEEVE BEFORE PRESSING

(*b*) PRESSED END OF CABLE WIRE

(*c*) SLEEVE PRESSED ON WITH THREE-PART DIE

(*d*) SECTION OF CABLE WIRE SPLICE WITH WIRES REMOVED FROM SLEEVE

FIG. 18.—DETAILS OF CABLE WIRE SPLICE.

them in the galvanized sleeve; and, finally, making the splice. Although this procedure is fundamental, the details underlying it differed from previous practice of making the splice by the method of drawing threaded ends together with a threaded sleeve.

In explanation of the method used, it should be pointed out that the result desired in the splice was one that would develop at least 95% of the specified minimum tensile strength of the wire and prevent failure due to untwisting. Acordingly, the wire ends were corrugated by pressure to give the necessary frictional resistance to the pull and, in addition, made elliptical to afford resistance against turning. The splice was completed by pressing a galvanized sleeve on the deformed wire ends. In the latter operation, the pressure was applied to the sleeve in two applications through a three-part die, the splice being turned through 60° after the first application. Fig. 18 shows details of the splice and Fig. 18(*d*) shows a section of a splice with parts dis-assembled.

There were 11 985 splice tests made. Slightly more than 4% failed to show the specified minimum value for strength, failure occurring in practically all cases by the wire pulling out of the sleeve. Based on the average ultimate strength and diameter obtained for the yield-point tests in Table 8 and the average load of 6 902 lb sustained by the splices, the average efficiency amounts to 98 per cent.

(5) Suspender Rope

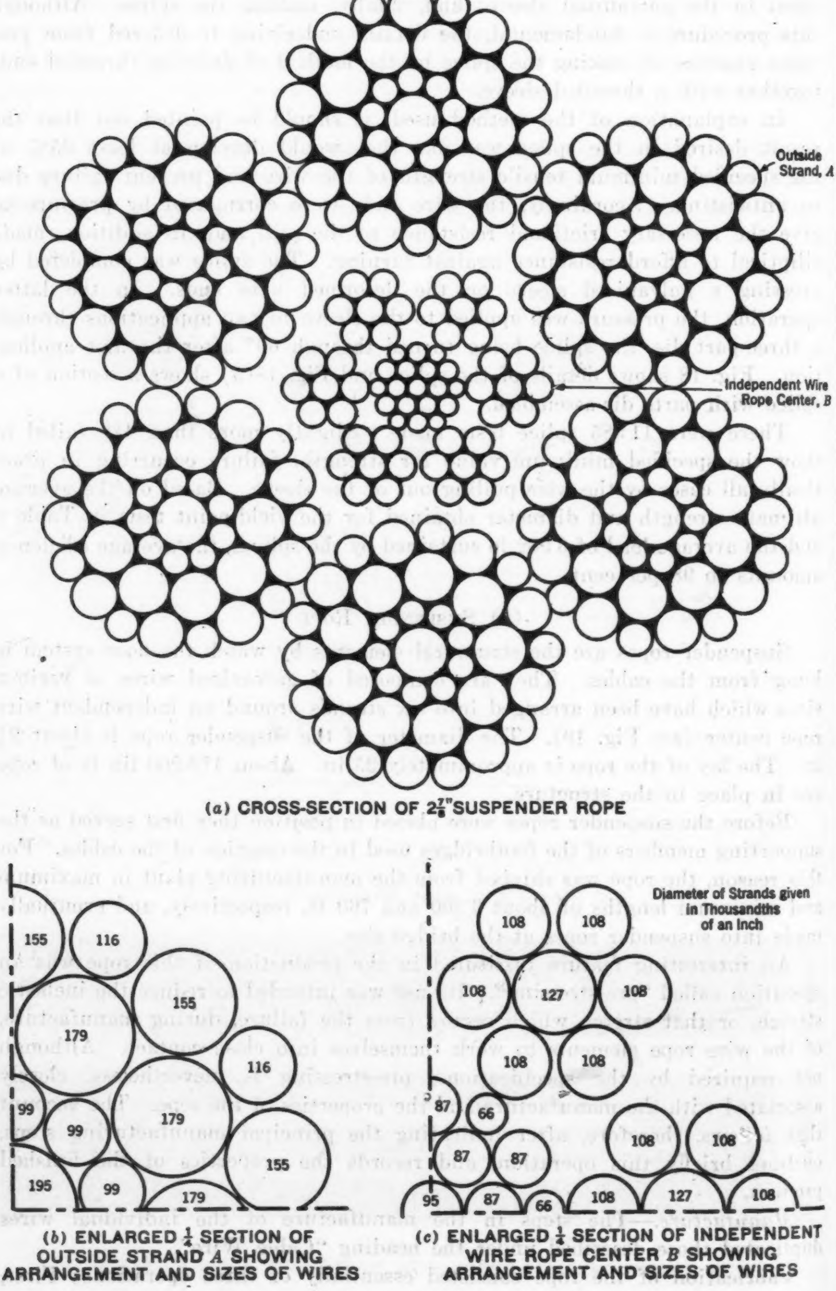
Suspender ropes are the structural elements by which the floor system is hung from the cables. They are composed of galvanized wires of various sizes which have been arranged into six strands around an independent wire rope center (see Fig. 19). The diameter of the suspender rope is about $2\frac{7}{8}$ in. The lay of the rope is approximately 25 in. About 170 200 lin ft of rope are in place in the structure.

Before the suspender ropes were placed in position they first served as the supporting members of the footbridges used in the erection of the cables. For this reason, the rope was shipped from the manufacturing plant in maximum and minimum lengths of about 3 500 and 760 ft, respectively, and eventually made into suspender ropes at the bridge site.

An interesting feature instituted in the production of this rope was an operation called "pre-stressing". Its use was intended to reduce the inelastic stretch, or that stretch which occurs from the failure, during manufacture, of the wire rope elements to work themselves into close contact. Although not required by the specifications, pre-stressing is, nevertheless, closely associated with the manufacture and the properties of the rope. The account that follows, therefore, after indicating the principal manufacturing steps, outlines briefly this operation, and records the properties of the finished product.

Manufacture.—The steps in the manufacture of the individual wires duplicated those described under the heading "Cable Wire".

Fabrication of the rope consisted essentially of three operations; First, making the strands from the individual wires; second, combining strands



(a) CROSS-SECTION OF $2\frac{7}{8}$" SUSPENDER ROPE

(b) ENLARGED $\frac{1}{4}$ SECTION OF OUTSIDE STRAND A SHOWING ARRANGEMENT AND SIZES OF WIRES

(c) ENLARGED $\frac{1}{4}$ SECTION OF INDEPENDENT WIRE ROPE CENTER B SHOWING ARRANGEMENT AND SIZES OF WIRES

FIG. 19.—CROSS-SECTION DIMENSIONS OF SUSPENDER ROPE.

to form the independent wire rope center; and, finally, closing the outside strands about the rope center. Strands were made in lengths sufficient to produce about 7 200 lin ft. of rope. To make strands of sufficient length, the individual wires were spliced by means of brazing, the customary practice.

Pre-Stressing Operation.—To subject the suspender rope to stress prior to its use as erection material was considered necessary by the manufacturer, in his scheme of erection. A description of the plans is, of course, outside the scope of this paper; but for the purpose of explaining the pre-stressing procedure there is pointed out a common characteristic of wire rope and the change to be effected in this product by the operation.

From the nature of its construction, the fact is evident that under load wire rope continues to elongate until the wires adjust themselves into a position at which the load can be resisted. As already referred to, it was desired to seat the elements of the rope among themselves through some system of loading so that the member would lose its inelastic stretch.

The apparatus used for the pre-stressing operation consisted of the following principal parts: (1) A length of railroad track for a car, upon which was mounted, horizontally, a sheave 8 ft in diameter; (2) at one end of the track, an hydraulic testing machine; and (3) at the other end of the track, two jacks, with appliances for holding the rope ends, one on either side of, and parallel to, the track.

The procedure was essentially as follows: One-half a manufactured length of rope was placed in position by attaching the ends to the jacks and passing a bight around the sheave. One end was pulled by a jack until load was registered by the testing machine to which the sheave-car was attached. The testing machine was then brought into play for raising the load to 400 000 lb and for sustaining this load for 10 hours. At the end of this time, the tension (200 000 lb) in the rope was released. Following this step in the operation, the rope was measured and cut to lengths required in the footbridge construction. The measurements for length were made while the tension was 80 000 lb, which was equalized in the two parts of the rope by manipulating jacks and testing machine.

The influence of the pre-stressing operation on the elastic properties of this rope may be observed from comparison of the characteristics of non-stressed and pre-stressed rope shown in Fig. 20. The theoretical cross-sectional area of the suspender rope is 4.0474 sq in. Points C and D, in Fig. 20, are the specified maximum elongations of 0.3 in. and 0.5 in. in a gauge length of 100 in., between loads of 5 000 and 205 000 lb per sq in., for pre-stressed rope and for rope not previously stressed, respectively. Test No. 1 was made of rope not previously stressed; the other tests were made on pre-stressed rope.

Tests and Physical Properties.—Tests were performed on the single wire lengths and on the rope itself. The former consisted particularly of the Preece test, although tensile tests were made on each coil of galvanized wire to insure attainment of a uniform raw material. The full-sized tests comprised two specimens cut from each manufactured length; one, simulating

a condition in the structure for obtaining the tensile strength as the rope lay over a sheave, producing a radius of 21 in. in the center line of the rope; and the other for the purpose of gauging the degree of stretch and observing the ultimate strength in a single-part test.

Sockets from among those made for use in the structure were attached to the ends of the rope specimens to aid in the testing.

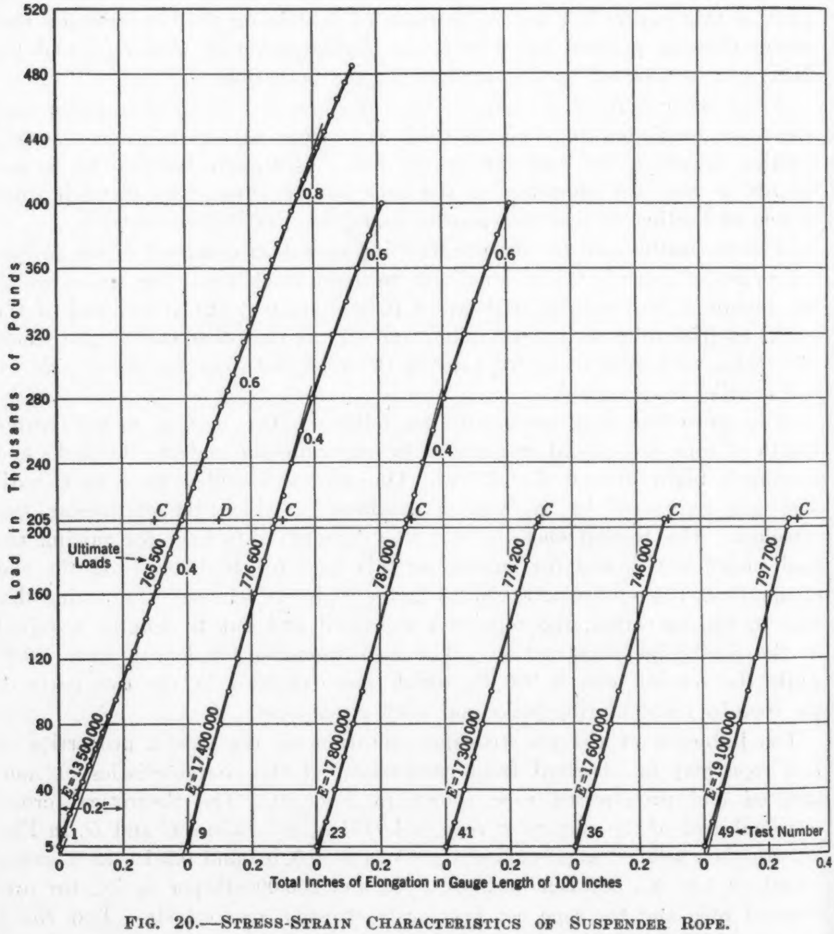


FIG. 20.—STRESS-STRAIN CHARACTERISTICS OF SUSPENDER ROPE.

For the full-sized tests, the specifications required that: (*a*) A rope tested over a sheave should develop a total strength in both parts of not less that 1 200 000 lb; (*b*) a rope tested straight should show, by strain measurement in a gauge length of 100 in., a stretch of not more than ½ in. from an initial load of 5 000 lb to one of 205 000 lb, for rope not previously loaded, nor more than 0.3 in. for rope after being subjected to repeated loading up

to 205 000 lb; and (c) the stretch of any rope should not vary more than 10% from the average stretch of all ropes tested. The results of the single and two-part tests are presented in Table 10.

TABLE 10.—RESULTS OF TESTS ON PRE-STRESSED SUSPENDER ROPE $2\frac{1}{8}$ INCHES IN DIAMETER

Reel No.	SINGLE PART TESTS: Ultimate strength, in pounds	Total elongation, in 100 in.,* in inches	Modulus of elasticity, in pounds per square inch	Double part tests, ultimate strength, in pounds
1	745 400	0.2940	18 600 000	1 328 000†
2–3	788 000	0.2760	18 900 000	1 369 200
4–5	770 000	0.2825	17 600 000	1 337 000
6–7	794 800	0.2995	17 300 000	1 361 400
8–9	776 600	0.2760	17 400 000	1 363 700
10–11	788 400	0.2960	17 900 000	1 359 500
12–13	777 000	0.2995	17 300 000	1 363 500
14–15	775 400	0.2875	18 000 000	1 362 700
16–17	788 600	0.2910	17 800 000	1 365 000
18–19	776 000	0.2900	18 200 000	1 312 700
20–21	772 800	0.2850	18 200 000	1 306 700
22–23	787 000	0.2850	17 500 000	1 380 800
24–25	780 000	0.2775	18 600 000	1 387 200
26–27	791 000	0.2830	18 200 000	1 304 000
28–29	786 200	0.2810	17 800 000	1 380 200
30–31	791 400	0.2880	17 400 000	1 373 300
32–33	774 200	0.2920	17 600 000	1 376 600
34–35	780 800	0.2835	17 900 000	1 356 200
36	746 000	0.2880	17 700 000	‡
37–38	766 000	0.2825	19 000 000	1 291 600
39–40	790 000	0.2895	17 700 000	1 367 200
41–42	774 200	0.2955	17 300 000	1 323 000
43–44	733 400	0.2835	17 600 000	1 242 800
45–46	777 100	0.2785	18 700 000	1 280 800
47–48	784 400	0.2850	18 500 000	1 244 200
49–50	797 700	0.2745	19 100 000	1 363 000
51–52	788 200	0.2845	18 100 000	1 285 400
53–54	798 200	0.2890	18 000 000	1 271 400
Average	778 500	0.2863	18 000 000	1 335 700

* Between 5 000 and 205 000 lb.
† Ultimate strength of rope in double part test not previously stressed.
‡ No double part test made.

(6) OTHER MATERIALS

In addition to the heat-treated eye-bars, structural and cast steels, cable wire, and suspender rope incorporated in the main steel structure and described heretofore, other kinds of metallic materials are: Heat-treated, forged, and rolled carbon steel; annealed forged carbon steel; rolled manganese-bronze and cast phosphor-bronze; and wrapping wire.

In the description that follows, the principal features concerning dimensions, quantity, manufacture, and properties are given consideration in regard to these materials.

Heat-Treated Steel Pins.—The heat-treated steel pins that connect the eye-bar chains to the girders embedded in the anchorages are 2 ft. 5.5 in. in length; there are two sizes, those in the New York anchorage being 10 in. in diameter and those in the New Jersey anchorage, $11\frac{1}{2}$ in. in diameter. In all, 100 pins are required for the two anchorages.

Briefly, the manufacturing operations in producing the pins consisted of forging, heat-treating, and machining. These operations were conducted in accordance with the best modern practice.

The properties of the pin material as determined by the ladle analyses and the acceptance tests are given in Table 11, as are also the requirements that the material had to meet.

TABLE 11.—Specified and Average Properties of Heat-Treated Pins and Bolts*

Part of structure	Number of tests	Physical: Yield point, in pounds per square inch	Physical: Tensile strength, in pounds per square inch	Physical: Percentage elongation, in 2 inches	Physical: Percentage reduction of area	Chemical: Carbon	Chemical: Manganese	Chemical: Phosphorus	Chemical: Silicon
Specified requirements..	....	60 000†	95 000†	21.0†	45.0†	.....	.....	0.04‡	0.05‡
Cable band bolts......	74	72 100	103 200	24	61	0.51	0.60	0.018	0.034
Eye-bar pins‡.........	9	63 600	101 300	25	56	0.51	0.65	0.023	0.029
Average..........	....	71 200	103 000	24	60	0.51	0.62	0.019	0.032

* Axis of specimen located midway between center and surface of member. † Minimum. ‡ Maximum.

The acceptance tests consisted of one tension and one bend test for each melt represented in each heat-treated lot. The specimens were of the usual diameter (0.5 in.) and were selected from points midway between the center and surface of the forgings. The bend specimen withstood a cold bend through 180° around a mandrel $\frac{1}{2}$ in. in diameter without showing signs of rupture on the outside of the bent part.

Heat-Treated Cable Band Bolts.—The finished bolt is $2\frac{3}{8}$ in. in diameter and 2 ft 3 in. in length under the head. A total of 3 400 bolts was required.

The manufacturing operations followed in sequence the steps of rolling, forging the head, heat-treatment, and machining. The billets were hot-rolled into bars of a nominal diameter of $2\frac{1}{2}$ in. The latter were cut into pieces long enough to permit forging the head. The resultant blanks were heated in a continuous furnace, quenched in water, and subsequently were drawn in a similar furnace.

The chemical and physical requirements that the heat-treated bolt material had to meet were the same as those specified for the heat-treated pin steel. (See Table 11.)

Specimen tests, one for tension and one for bend, were made for each heat-treated lot of fifty bolts. The bend specimens withstood a 180° flat bend. The average results of the specimen tension tests and the average results of the ladle analyses are shown in Table 11. In addition to the specimen tests, an occasional full-sized bolt was tested in tension. These full-sized tests (ten in number), indicated a range from 66 000 to 85 000 lb sq in. for the yield point, and from 109 000 to 127 900 lb per sq in. for the tensile strength, the average being 73 920 and 116 900 lb per sq in., respectively. Failure in each case occurred in the threads.

The stress-strain characteristics shown graphically in Fig. 21 are typical of the material. The five melts of steel from which the required number of

bolts were made, are there represented; one of them (Melt *E*, Fig. 21) is of Mayari or chrome-nickel steel. Specimens were cut from a point midway between the center and surface of the heat-treated, unfinished bolt, and were machined to a diameter of 0.505 in. The dotted extensions to the curves in Fig. 21 indicate excessive stretch (not less than 0.009 in. per in.) that occurred before the next load increment had been applied. The yield point

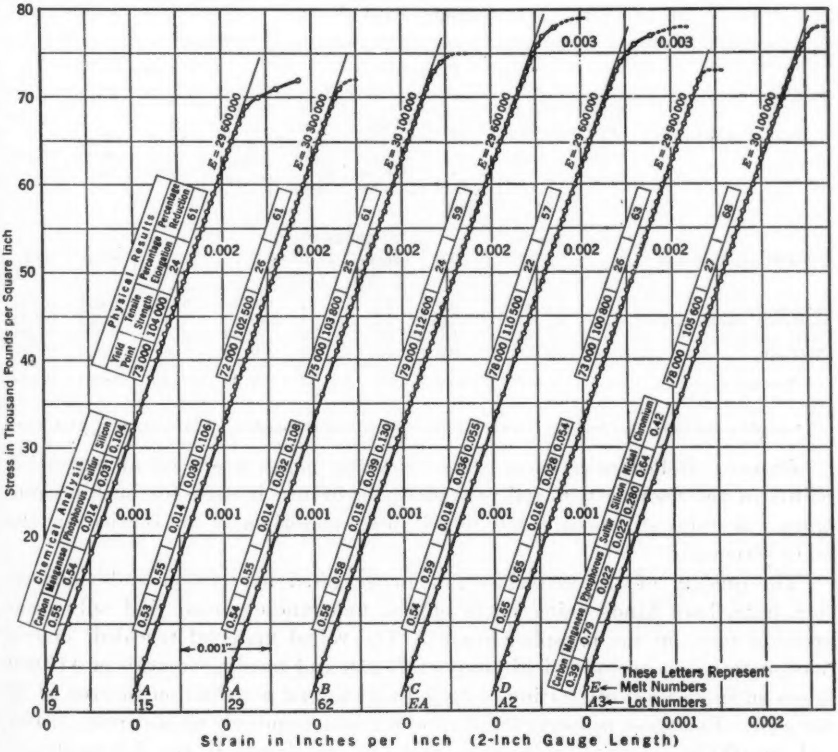


FIG. 21.—STRESS-STRAIN CHARACTERISTICS OF HEAT-TREATED CABLE BAND BOLTS.

(the drop of the beam), was considered when plotting the broken line. Samples for chemical analysis were milled from the broken tension specimens.

Annealed Steel Pins, Rollers, and Rockers.—The number of annealed forged pins in the eye-bar chains in the anchorages is 992; of rollers under the tower cable saddles, 328; of rockers under the anchorage cable saddles, 120; and of miscellaneous pins in the floor system, 14. The anchorage eye-bar pins are 10 in. in diameter and from 12 in. to 2 ft $5\frac{1}{2}$ in. in length; the rollers, 8 in. in diameter and 8 ft 6 in. in length; the rockers, 12 in. in diameter, but with two parallel sides giving a 7-in. width, and 7 ft in length; and the floor system pins are of various diameters and lengths. The manufacturing operations on these members were similar to those outlined in this paper for the heat-treated pins except that they were annealed.

This material was manufactured in accordance with the Specifications for Class E Annealed Carbon Steel Forgings of the American Society for Testing Materials (Serial Designation A18-27). The properties of the annealed material are given in Table 12. The stress-strain curves of three specimen tests representing rockers are plotted in Fig. 3.

TABLE 12.—SPECIFIED AND AVERAGE PROPERTIES OF ANNEALED FORGED CARBON STEEL*

Part of structure	Number of tests	PHYSICAL				CHEMICAL			
		Yield point, in pounds per square inch	Tensile strength, in pounds per square inch	Percentage elongation, in 2 inches	Percentage reduction of area	Carbon	Manganese	Phosphorus	Silicon
Specified requirements...	....	Tensile strength / 2	75 000†	1 725 000 / Tensile strength	2 640 000 / Tensile strength	.....	0.40 to 0.80	0.05‡	0.05‡
Eye-bar pins...........	33	40 600	78 000	27	44	0.38	0.53	0.024	0.034
Anchorage saddle rockers	6	43 800	82 500	25	41	0.39	0.66	0.014	0.035
Tower saddle rollers....	26	44 300	78 200	28	43	0.39	0.59	0.018	0.034
Other pins.............	3	48 200	86 100	25	42	0.44	0.63	0.026	0.035
Average..........	....	42 700	78 800	27	43	0.39	0.59	0.020	0.034

* Axis of specimen located midway between center and surface of member. † Minimum. ‡ Maximum

Bronze.—Rolled manganese-bronze wearing plates are used at expansion points in the floor system and cast phosphor-bronze is used for bushing pin-holes. A total of about 6 450 lb of these materials is in service in the main structure.

The quality of the materials was investigated by tension and compression tests, both kinds being made on the manganese-bronze, and only compression tests on the phosphor-bronze. The rolled material revealed, in two tension tests, an average yield point of 77 250 and tensile strength of 112 000 lb per sq in., a total elongation of 18% in 2 in., and a reduction in area of 22 per cent. For these properties the specifications required, respectively, 55 000 and 100 000 lb per sq in. minimum, and 15% minimum in the deformations. The specified properties and average results of the compression tests and chemical analysis of this material are given in Table 13.

Wrapping Wire.—The material with which the cables are wrapped is galvanized wire about 0.148 in. in diameter. Approximately 409 tons of this wire were required.

The steel was made by the basic open-hearth process and is of the following average analysis: Carbon, 0.10; manganese, 0.50; phosphorus, 0.020; and sulfur, 0.030. The material was manufactured essentially in the same manner as the cable wire.

The quality of the wire was investigated by the tension test and the quality of the galvanizing by the Preece test. The tensile strength and total elongation were measured, although the specifications required only the determination of elongation in 10 in., the minimum value for which had to be

10 per cent. The average tensile strength of the wire is about 68 000 lb per sq in., based on the gross cross-sectional area, which includes the zinc coat; the average elongation proved to be about 16 per cent. One end of each coil was tested for tensile strength and elongation, and 5% of the coils were tested for quality of galvanizing.

TABLE 13.—SPECIFIED PROPERTIES AND AVERAGE RESULTS OF COMPRESSION TESTS ON ROLLED MANAGANESE-BRONZE AND CAST PHOSPHOR-BRONZE

Item No.	Description	Number of tests	PHYSICAL PROPERTIES: Load at permanent set of 0.001 in., in thousands of pounds	PHYSICAL PROPERTIES: Permanent set at load of 100 000 lb per sq in., in inches	CHEMICAL PROPERTIES: Copper	Zinc	Manganese	Phosphorus	Iron	Aluminum	Lead	Tin	Remainder
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
ROLLED MANGANESE-BRONZE; SPECIMEN, 0.75 INCH BY 1.33 INCHES BY 1.00 INCH													
1.....	Specified requirement..	....	50.0*	0.05†	62 to 68	23 to 29	....	....	....	....	....	....	.....
2.....	Average......	2	58.7	0.024	67.56	20.79	3.72	0.025	1.18	6.06	0.66	....	.....
CAST PHOSPHOR-BRONZE; SPECIMEN, 1.128 INCHES IN DIAMETER BY 1.00 INCH IN LENGTH													
3.....	Specified requirement..	....	19.0*	0.25†	81.5*	.....	....	1.0†	....	....	....	17.0†	0.5†
4.....	Average......	16	24.6	0.12	85.73	.....	....	0.64	....	....	....	13.55	0.08

* Minimum. † Maximum.

The coils (each of which weighed about 175 lb) were spliced to form long lengths for shipment to the bridge site. The splice was made by butt-welding and had an efficiency of about 98 per cent. In most cases, the weld-test specimen ruptured a short distance from the junction of the two wires.

TABLE 14.—MATERIALS IN THE MAIN STEEL STRUCTURE, IN TONS

Item No. (1)	Materia (2)	Anchorages (3)	Towers (4)	Cables (5)	Suspended structure (6)	Total (7)
	Heat-Treated Steel:					
1	Eye-bars..........	4 481	....	....	....	4 594 (items 1–3)
2	Steel pins..........	36	....	....	....	
3	Steel bolts..........	....	....	77	....	
	Structural Steel:					
4	Carbon..........	1 825*	18 254	36	8 869	28 984
5	Silicon..........	....	23 587	....	8 132	31 719
6	Cast steel..........	397†	1 450§	728	68	2 643
7	Annealed forged steel..........	342‡	238¶	....	3‖	583
8	Cable wire..........	....	....	28 308	....	28 308
9	Wrapping wire..........	....	....	409	....	409
10	Suspender rope..........	....	....	1 234**	....	1 234
11	Hand ropes..........	....	....	43††	....	43
12	Reinforcement (bulb-beams and tie-rods in roadway)..........	....	....	....	2 341	2 341
13	Bronze..........	....	....	....	3	3
	Miscellaneous:					
14	Railings..........	....	....	....	636	636
15	Conduits, light standards, etc....	....	....	....	261	261
16	Total..........	7 081	43 529	30 835	20 313	101 758

* Excluding anchorage floor system. † Anchorage saddles and strand shoes. ‡ Pins and rockers § Tower saddles. ¶ Rollers. ‖ Pins. ** 14.5 lb per lin ft. †† 2.2 lb per lin ft.

(7) Distribution of Materials in Main Divisions of Structure

Approximately 100 000 tons of metallic materials were required in the construction of the main bridge structure not including the approaches. The quantity of each kind of material in each of the four principal divisions of the main steel superstructure is given in Table 14. The quantities given do not include the floor system in the anchorages, or the concrete reinforcing rods.

Summary

The writer wishes to emphasize again the fact that the materials of the structure and the methods of manufacture are in no way experimental, but in every case are representative of the best modern practice. In dealing with the subject-matter of this paper he has intentionally treated briefly, or has omitted entirely, a detailed description of such phases of the shop work as are met with in every-day common practice, even though such operations are of interest to those unfamiliar with the methods of manufacture. Feeling that such descriptions may be found elsewhere, and are not properly a part of this paper, he has rather attempted to indicate the conformity of the work with the most modern manufacturing methods and also with the specifications, and to point out such modifications in requirements of materials or in methods as are applicable to this work. The program of tests and the characteristics of materials incorporated in the structure are treated in some detail because it is believed that, in view of the magnitude of the operation and the care taken in following a comprehensive testing program, this phase of the subject will be of special value to the Engineering Profession.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

GEORGE WASHINGTON BRIDGE: APPROACHES AND HIGHWAY CONNECTIONS

By J. C. Evans,[1] Esq.

Synopsis

The approaches to the George Washington Bridge, including an outline of the steps in the development of the plans from their simplified beginnings, are described in this paper. It contains a discussion of the construction of the New York approach in stages to conform to traffic requirements and financial limitations.

Radically different conditions were met at the two ends of the bridge. In Manhattan, the approaches are in a highly congested area, while in New Jersey they are in a suburban territory. The history of the development of the plans for these approaches indicates the great amount of time and effort required to arrive at satisfactory solutions of the problems. The close co-operation which was sought and obtained between the Port Authority, as the constructing body, and the Municipal, County, and State Governments, was an important factor in the work.

Location of Bridge to Meet Traffic Needs

The location of the bridge at Fort Washington Park, between 178th Street and 179th Street, in Manhattan, and at Fort Lee, N. J., fills a definite and pressing traffic need of the Metropolitan Area. The bridge is ideally situated for traffic from the south and southwest bound to and from points in New England. At this location such traffic may avoid the congested area of Lower Manhattan. The bridge is also well situated to serve the needs of a rapidly growing suburban area of Northern New Jersey. In recent years, Upper Manhattan and the contiguous areas have been growing in population at a rapid rate, and this territory also demands facilities for access to the New Jersey area.

At the bridge site, Manhattan Island is roughly $1\frac{1}{4}$ miles wide between the Hudson and the Harlem Rivers. Riverside Drive skirts the Hudson River and Fort Washington Park and bounds this territory on the west,

Note.—Discussion on this paper will be closed in May, 1933, *Proceedings*.

[1] Terminal Engr., The Port of New York Authority, New York, N. Y.

while the Harlem River Speedway skirts it on the east. Between these prominent highways are several important north and south arteries, the most important of which are, from west to east, Fort Washington Avenue, Broadway, St. Nicholas Avenue, and Amsterdam Avenue. Almost directly east of the site of the George Washington Bridge, the Harlem River is crossed at 181st Street by the Washington Bridge which connects with University Avenue in The Bronx and thence with all other principal arteries of that area.

In New Jersey at the time the bridge location was chosen no principal modern traffic artery passed through Fort Lee. The Borough itself is a comparatively small community of 9 000 population. It had three fairly good highways—Lemoine Avenue leading north in the direction of Englewood, Palisade Avenue leading south, and Fort Lee Turnpike leading west to Hackensack and the Paterson and Passaic area—but in the sense of the modern highway, it had none. The fine modern highways that, when completed, will focus upon the bridge and carry its traffic by direct routes to all parts of the New Jersey area, are due to the progressive and co-operative attitude of the New Jersey State Highway Commission that has provided the means by which the full utility of the bridge can be realized in the New Jersey territory.

Approaches as Major Elements of the Bridge Project

The choice of the exact location of the structure was determined largely by construction considerations, such as span and foundation conditions. The legislation directing the Port Authority to construct the bridge specified a location between 170th Street and 185th Street, in Manhattan, and a point approximately opposite thereto in the Borough of Fort Lee in New Jersey. Within these limits three sites were studied. The final choice was dictated largely by the fact that studies indicated the location between 178th Street and 179th Street as the most desirable with respect to approaches, grades, and street connections. Fortunately, the studies also indicated the location chosen to be the most economical from a construction standpoint. In New Jersey, the location was such as to permit the development of the approach with its greater space requirements for toll collection in a part of the territory north of the main business center of Fort Lee, thus utilizing less expensive property and offering the minimum of interference to local development, and, at the same time, giving maximum accessibility to the business district and to the principal traffic arteries converging in this locality.

The main bridge structure fulfills its purpose only to the extent that the approaches are able to distribute the traffic adequately. This is especially true in the case of an approach in Upper Manhattan where the traffic arteries were already carrying a high percentage of their estimated capacities at the time of the opening of the George Washington Bridge. The actual bridge roadway opened to traffic was limited accordingly, under initial conditions, to four lanes—the maximum amount which the municipal authorities believed could be handled adequately by the connections.

The old conception of a bridge approach as a simple ramp from the main structure to the ground surface, terminating in a plaza, has required considerable revision under such circumstances. There have been diverse views as to what constitute approaches, and the Port Authority, through its extended negotiations with the municipalities, has adopted a broad viewpoint to the effect that the approaches should not merely embrace the ramps leading from the bridge proper to wherever they may strike the ground, but that they should also include adequate connections between the ramps and such existing through arteries as will assure the proper flow and distribution of the bridge traffic to such arteries. It was recognized that the municipalities should not be burdened with the cost of such necessary connections when they are to carry predominantly through traffic over the bridge. On the other hand, the construction by the Port Authority of adequate connections to assure the unhindered maximum flow of traffic to and from the bridge is unquestionably in conformity with sound business policy and in protection of the bond-holder's interests. Ramps or tunnels communicating directly with arteries of adequate capacity have thus been incorporated in the approach plans.

In New Jersey, the development of the approach plans was such as to make it extremely difficult to say where the bridge approach should end and the State highway system begin, the bridge itself becoming in fact an extension of a high-speed ultra-modern State highway system.

The relation between the monies expended in developing the approach facilities and the cost of construction of the main structure gives an indication of the major importance of the approach elements. The cost of the approaches accounts for about 21% of the total sum expended to date (1933) on construction by the Port Authority. Including real estate, engineering, administration, and expenses of financing, the approaches account for approximately 37% of the expenditures of the Port Authority on the project. If, in addition, the cost of the new roadways constructed in New Jersey by the State Highway Commission directly for the benefit of bridge traffic is considered, the total cost of approach and approach highway connections is well over 40% of the total cost of the facility.

Approval of Plans by Governors and Municipalities

The statutes authorizing the construction of the bridge provided that the plans for the approaches should be subject to the approval of the Governors of the two States and of the respective municipalities in which the facilities were to be located. Such an arrangement is only proper and the Port Authority early sought the counsel and approval of the officials of the City of New York and the Borough of Fort Lee, as well as the approval of all other county and State highway bodies interested in the bridge project.

The approach plans for both sides of the river, as they have been adopted finally, embody largely the ideas of the officials who have acted with representatives of the Port Authority on joint committees appointed to develop the plans. As presented in the early studies, the approaches underwent

repeated modifications and improvements; perhaps no better illustration can be found of the advantage of the continuous development of a problem of such intricate character over a period of a number of months, requiring as it does complete co-operation between different interests. It would have been impossible to secure as satisfactory results in this work had it been necessary to prepare complete plans before contracting for any work on the structure.

Development of the New York Approach Plan

One of the early proposals for the New York approach provided for its construction in two stages. In the initial stage the approaches were to occupy only the blocks between 178th and 179th Streets, from Haven Avenue to Fort Washington Avenue, with a plaza between these two streets from Fort Washington Avenue to Broadway, as shown on Fig. 1. It was believed at that time that the initial stage as proposed would make adequate provision for the bridge traffic for a period of five to eight years following completion of the initial stage in 1932. During this period bridge traffic to and east of Broadway could be accommodated by widening West 178th and West 179th Streets, between Fort Washington Avenue and Broadway, with crossing of bridge traffic and street traffic at grade on Fort Washington Avenue. It was intended that traffic to and from Riverside Drive would utilize 178th and 179th Streets, and existing connecting streets west of Fort Washington Avenue.

The ultimate stage as then planned is shown on Fig. 2, and indicates a continuation of the boulevard treatment from Broadway to Amsterdam Avenue, connecting the approaches to the George Weshington Bridge with the approaches to the Washington Bridge over the Harlem River. The boulevard provided for the crossing of intersecting streets at grade, except Broadway, which thoroughfare was to be depressed under the boulevard, and a traffic circle established in order to avoid interference of bridge traffic by or with through Broadway traffic.

The entire question of location and approach plans was submitted to the City Officials for consideration as soon as the arrangements for financing were completed. The mutual interests of the City and the Port Authority in the bridge project were considered by a Joint Conference Committee of City and Port Authority representatives. At a meeting of this Committee in January, 1927, an Engineering Sub-Committee was appointed, consisting of the Chief Engineers of the Board of Estimate and Apportionment, the Borough of Manhattan, the Board of Transportation, the Department of Plant and Structures, the Dock Department, and representatives of the Park Department and the Department of Finance, for the City, and the Chief Engineer and two Consulting Engineers, representing the Port Authority. This Committee had before it decision as to the site for the structure in Fort Washington Park and plans for the approaches.

At first, it was the feeling of the municipal representatives on the Committee that the question of approach plans and approval by the City, of the bridge site, were so bound together that decision on one could not be made

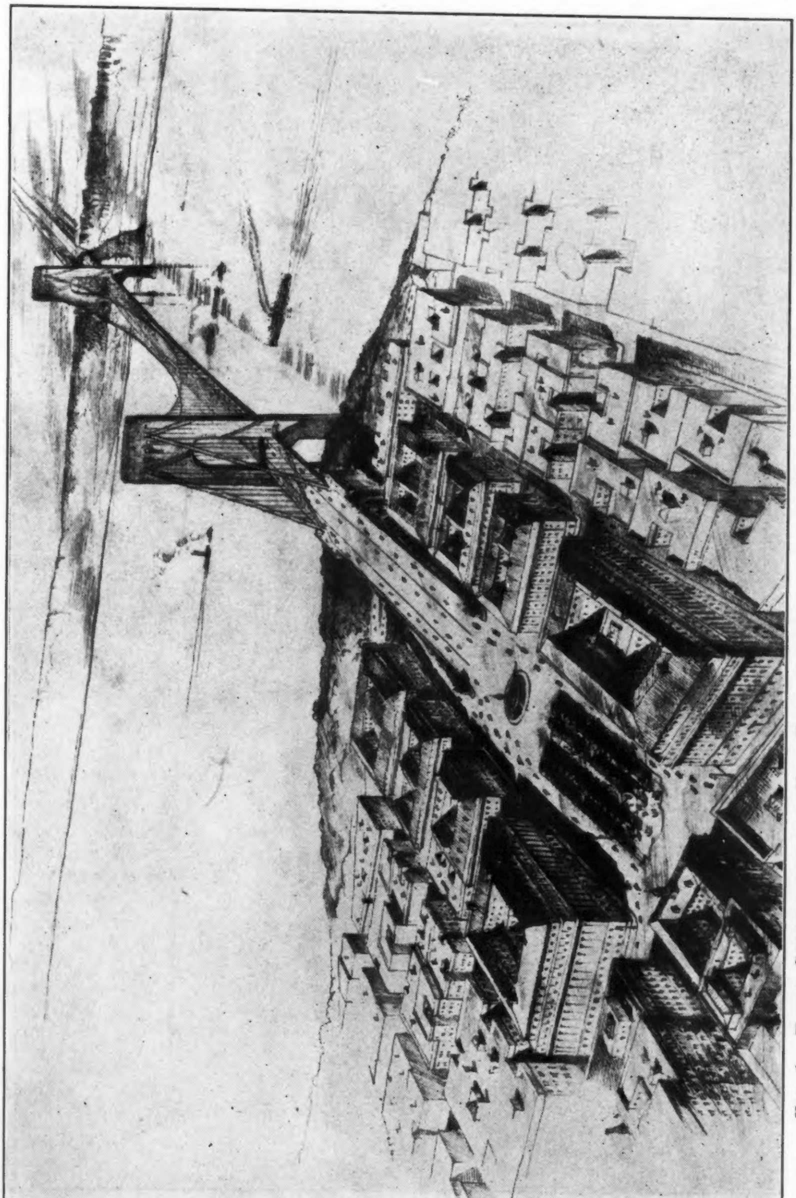


FIG. 1.—EARLY SCHEME FOR THE NEW YORK APPROACH, GEORGE WASHINGTON BRIDGE (INITIAL STAGE).

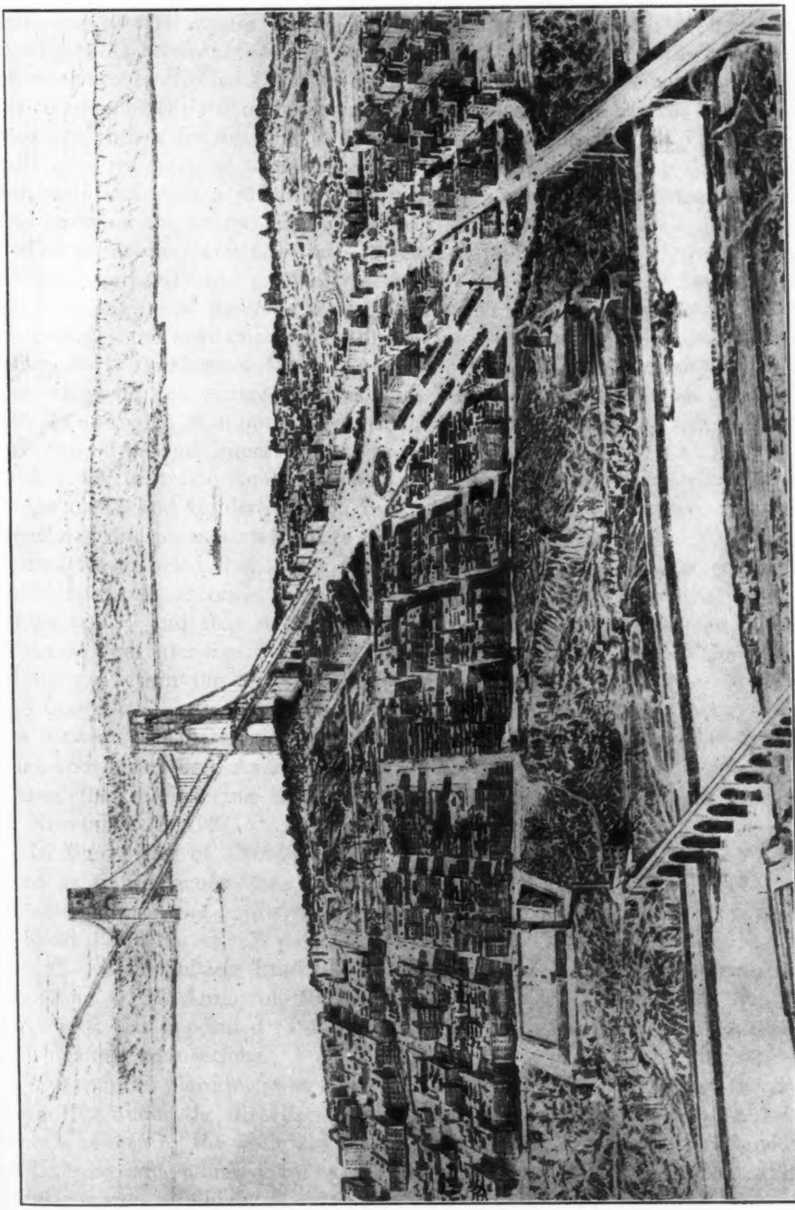


FIG. 2.—EARLY SCHEME FOR NEW YORK APPROACH, GEORGE WASHINGTON BRIDGE (ULTIMATE STAGE).

without, at the same time, reaching decision on the other. However, in March, 1927, it became apparent that a comprehensive plan of street connections and arterial routes should be developed without regard to terms and conditions as between the Port Authority and the City. It was likely that the work would involve considerable time and expense inasmuch as the plan was to be sufficiently comprehensive and elaborate to take care of future conditions within a considerable radius of the bridge plaza. The City accordingly gave its approval to the bridge location without requiring the complete elaboration of such a plan and the working out of an agreement with the City covering the bridge approach.

The discussions and negotiations between representatives of the City and the Port Authority had gradually developed the viewpoint that future facilities to take care of the growth of bridge traffic should provide main outlets, other than those available by existing surface street connections, to Riverside Drive, Fort Washington Avenue, and Broadway. It was generally conceded that whenever the bridge traffic should increase to a volume of 10 000 000 vehicles annually, such outlets—particularly an outlet to points east of Broadway—would become imperative.

A study of traffic conditions was made by a special committee. Future bridge traffic and its distribution in the vicinity of the approach, the street capacities, and present and future local traffic were all analyzed. The Special Committee reported that under the original plan the approach street system would be taxed at once to or beyond its capacity by the addition of the bridge traffic, and that within approximately four years after the opening of the bridge extensive deficiencies in capacity would prevail on practically all streets within the affected area, with the exception of Riverside Drive. The Committee recommended, among other things, increased roadway facilities between the plaza and Riverside Drive, and a new facility between the plaza and Amsterdam Avenue. In line with the findings of the Special Committee, the Engineering Sub-Committee reported similar recommendations on November 11, 1927.

In the spring of 1928 the Port Authority and the City reached agreement as to the general construction of approaches to Riverside Drive and an extension of the approach to Amsterdam Avenue by means of a tunnel in West 178th Street. A committee composed of Arthur S. Tuttle, M. Am. Soc. C. E., Consulting Engineer of the Board of Estimate and Apportionment and O. H. Ammann, M. Am. Soc. C. E., Chief Engineer of the Port Authority, was appointed to develop detailed plans for the bridge approaches and highway connections.

The actual planning was done by the Engineering Staff of the Port Authority under the direction of the aforementioned Committee. A set of criteria governing the work was drawn up to meet the requirements of modern traffic, and was adhered to as far as was reasonably practicable. These criteria were as follows:

1.—An effort should be made to decentralize traffic lanes on the approaches, thus avoiding one central locus of congestion at a "bridgehead."

2.—There should be automatic traffic routing and control, rather than a complicated mechanical or electrical installation or an expensive personnel to direct bridge traffic.

3.—No crossings at grade of intercepting traffic, whether of street or bridge traffic, should be permitted.

4.—Left turns should be avoided.

5.—Provisions for toll collection should be eliminated on the New York approach, all tolls to be collected in New Jersey.

6.—All grades should be as flat as possible consistent with drainage and, in any event, should not exceed 4% on main approach and connections carrying truck traffic, and 6% on connections with Riverside Drive carrying passenger automobiles only.

7.—As far as possible any approach roadway should accommodate "one-way" traffic only.

8.—An approach roadway handling bridge traffic exclusively should be widened and banked on curves to allow for normal traffic speed.

9.—Bridge traffic should be separated from normal street traffic until it reached a point of connection with a through thoroughfare.

10.—In revising existing street lines or grades, an effort should be made to confine the damage to abutting property to a minimum.

11.—Construction should be in two stages, that for the initial stage to be such as to make possible its incorporation in the later stages without interruption of traffic during the later construction.

12.—The street map of Manhattan should be revised or changed no more than necessary.

13.—Bridge traffic should not be added to streets carrying an appreciable amount of ordinary street traffic.

14.—All new construction of roadways should be built for the ultimate capacity of the bridge.

The Committee presented two reports; the first, on April 4, 1929, proposed a plan essentially like that ultimately followed, but mentioned the possibility of constructing a depressed roadway instead of a tunnel for the connection with Amsterdam Avenue. The Committee recommended that the cross-town roadway be built when traffic should reach 10 000 000 vehicles annually and funds should become available. The latter stipulation was made because of the limited amount of funds available from the original financing, the original New York approach plans having contemplated nothing beyond the plaza at Fort Washington Avenue.

The City recognized the position of the Port Authority, but was so impressed with the necessity of having the tunnel approach ready at the time of opening the bridge that it decided to arrange to finance and build the tunnel, with the agreement that the Port Authority would accept it when traffic should reach 10 000 000 vehicles annually, but not later than three years after the completion of the tunnel, when the Port Authority would pay the cost of the tunnel, not to exceed $2 500 000.

The Committee of Engineers continued its work and prepared detailed plans and estimates for a four-lane tunnel. These plans developed the fact that the tunnel would require an expensive ventilation system and would cost approximately $6 000 000, much more than had been anticipated. It was out of the question for the City to proceed with the work at that figure.

Studies were then made of an open depressed roadway, which had been mentioned as an alternative in the report of April 4, 1929, of the Engineering Committee. The great disadvantage of the depressed roadway was the amount of property required. The City objected to the destruction of so much property value.

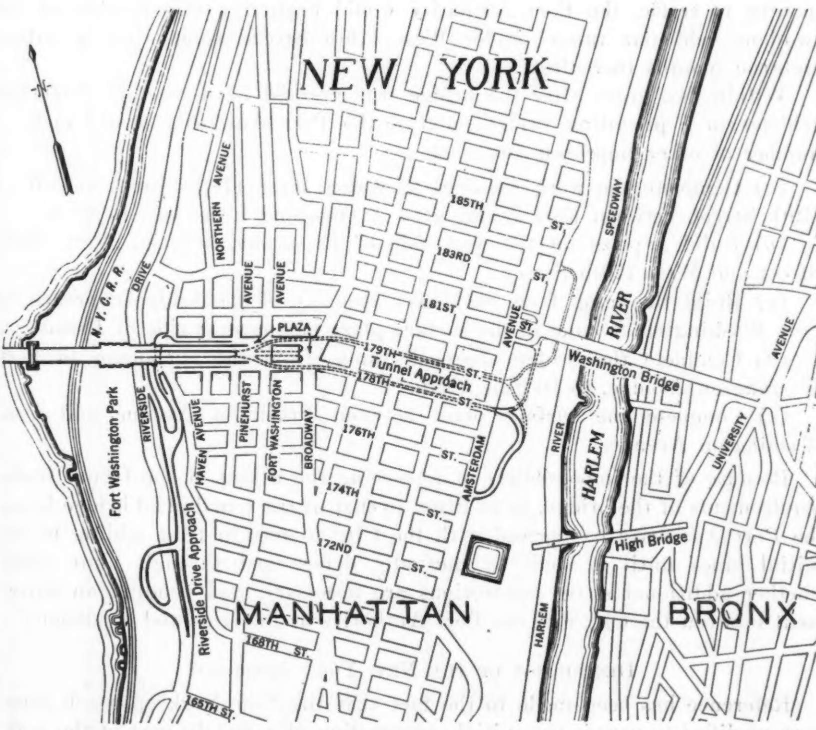


FIG. 3.—GENERAL PLAN OF NEW YORK APPROACH AND CONNECTIONS.

After much study of the situation, the Committee rendered a report on March 17, 1930, recommending a method of procedure which would provide the necessary facilities. In brief, it was proposed that for the initial traffic condition the Port Authority construct a two-lane, two-way tunnel in West 178th Street and widen West 178th and West 179th Streets, between Fort Washington Avenue and Broadway, to 36 ft by setting back the curbs. For the final traffic condition a second two-way tunnel was to be constructed in West 179th Street. The Port Authority agreed to the plan (Fig. 3) and further agreed to complete the approach roadways between Fort Washington

Avenue and Haven Avenue and the Riverside Drive connections west of Haven Avenue for the initial traffic condition.

In order to make it financially feasible for the Port Authority to build the tunnel under 178th Street, certain modifications in the procedure for carrying out the plan for the facilities west of Broadway were required. A plan was devised whereby approximately \$4 500 000 worth of work could be deferred, to be done later as funds should become available, and well in advance of the traffic requirements. This procedure would relieve the City wholly of the necessity of financing any part of the work.

The plan provided that after the bridge should have been opened six months to traffic, the Port Authority would begin the construction of the two-lane vehicular tunnel under West 179th Street, completing it within eighteen months thereafter.

Within two years after the bridge traffic would have reached 10 000 000 vehicles in a preceding twelve months, the Port Authority would make a number of other improvements, such as:

(*a*) Complete the work required to widen West 178th Street and West 179th Street, between Fort Washington Avenue and Broadway, to 80 ft.

(*b*) Build a plaza on the west side of Broadway, between West 178th Street and West 179th Street.

(*c*) Build the ramp from the lower roadway of the bridge approach at Fort Washington Avenue to the surface plaza on the west side of Broadway.

(*d*) Complete the bridge approach ramp, from the anchorage to Fort Washington Avenue, to its full width.

(*e*) Complete the surface plaza between Pinehurst Avenue and Fort Washington Avenue.

Because of the impossibility of foreseeing the extent of the future traffic requirements of the bridge, in addition to that of the four initial bridge lanes, the Port Authority has agreed with the City to refrain from adding to the initial lanes until it shall be mutually determined through joint study whether additional street connections are necessary, and pending an agreement between the City and the Port Authority as to terms and conditions.

Description of the New York Approach

Reference has been made to the fact that the New York approach plans were modified to permit the initial construction of a certain part of the work prior to the opening of the George Washington Bridge to traffic and the remainder of the work necessary to complete the bridge approaches, according to the plan approved by the Board of Estimate and Apportionment of the City of New York, the Port Authority, and the Governor of the State of New York, at a later date.

The plan may be better understood by reference to Figs. 4 and 5. It embraces the following principal elements:

1.—The main approach ramps from the bridge proper to the plaza area west of Fort Washington Avenue and the side ramps and street connections beside the main ramp as far west as Haven Avenue.

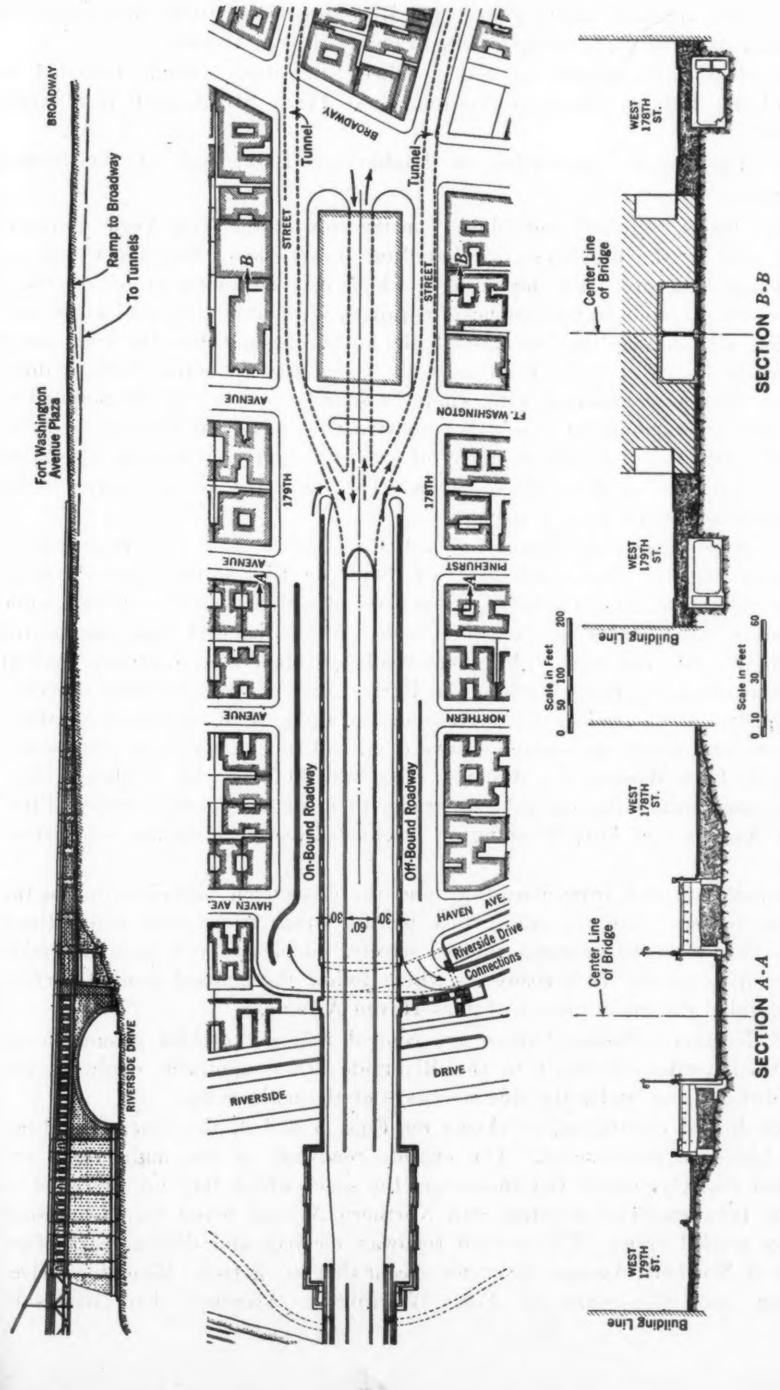


FIG. 4.—NEW YORK APPROACH, GEORGE WASHINGTON BRIDGE, ANCHORAGE TO BROADWAY, FINAL CONDITION.

2.—The approach connections with Riverside Drive from their points of junction with the side ramps or roadways at Haven Avenue.

3.—The improvements in the so-called "Broadway Block" bounded by Broadway, Fort Washington Avenue, West 178th Street, and West 179th Street.

4.—The tunnel approaches to Highbridge Park, east of Amsterdam Avenue.

The terms, "initial" and "final" construction of the New York approach, refer only to the provisions for handling the traffic of the upper deck of the main structure. The lower deck, which may be added to accommodate four lines of rapid transit traffic if required, at a later date, will make connection with a ramp passing through the anchorage and over the arch across Riverside Drive at a level immediately below the vehicular traffic ramps, and so into its connection with whatever subway system may be planned at the time. It should be noted that construction has been planned so as to permit suitable future development of such an approach without alteration to the supports of the surface ramps. The plan will be considered under the aforementioned four headings.

1.—Main Approach Ramps to Fort Washington Avenue.—According to the plan for the "final condition," as shown on Fig. 4, the approach ramp is separated into three roadways, the central of which is 60 ft in width, while the roadways on either side are 30 ft wide. Sidewalks, 6 ft wide, are located beside the side roadways. All three roadways start at a common level at the anchorage in Fort Washington Park. The central roadway descends eastward on a 4% grade, attaining the level of the ground surface at Northern Avenue, and continues thence eastward on a 1.82% grade to a sub-surface plaza at Fort Washington Avenue. The two 30-ft outside roadways slope down more gradually, on a 1.2% grade, to a surface plaza between Pinehurst Avenue and Fort Washington Avenue, directly above the sub-surface plaza.

Vehicles to and from Broadway and the cross-town tunnels will use the central roadway and the sub-surface plaza. Other traffic may make direct connection with the adjacent surface streets, vehicles to and from Riverside Drive utilizing the 30-ft roadways which follow the general ground surface and parallel the main ramp as far as Haven Avenue.

At Northern Avenue, where the central roadway attains ground level, direct connections from it to the Riverside Drive approach roadways are provided, passing under the side roadways of the main ramp.

For initial conditions, as shown on Figs. 5 and 6, the central roadway only has been constructed. The outside roadways of the main ramp are omitted entirely east of the anchorage, the space which they are designed to occupy between Service Street and Northern Avenue being for the present merely graded areas. The central roadway receives and discharges surface traffic at Northern Avenue, all traffic except that to, or from, Riverside Drive, passing over the plaza at Fort Washington Avenue. The Riverside

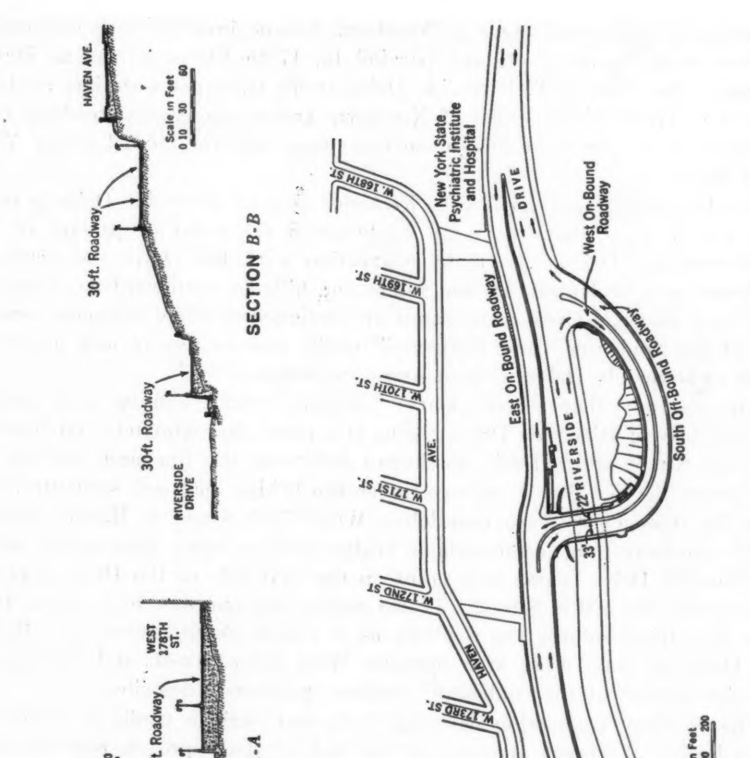


FIG. 5.—NEW YORK APPROACH, INITIAL CONDITION, INCLUDING RIVERSIDE DRIVE CONNECTIONS.

Drive traffic "offbound" turns at Northern Avenue into the 30-ft connecting roadway running north of, and parallel to, 178th Street as far as Haven Avenue. The "onbound" Riverside Drive traffic traverses a similar roadway parallel to 179th Street. East of Northern Avenue the central roadway continues to an entrance to the sub-surface plaza and the tunnel under West 178th Street.

2.—Approach Connections with Riverside Drive.—Riverside Drive is more than 115 ft lower than the ramp roadways at the point of passage of the latter over the Drive. To make connection with the Drive the approach roadways west of Haven Avenue follow the hillside southward as shown on Figs. 5, 7, and 8. These connections are designed to effect complete separation of the "onbound" and "offbound" traffic and to receive and discharge traffic at Riverside Drive without grade crossings.

The roadway that accommodates "onbound' traffic coming from points south by way of Riverside Drive begins at a point approximately 200 ft south of 168th Street and extends northward following the line and gradient of the former Service Street, crosses under the bridge approach structure, and joins the approach roadway paralleling West 179th Street at Haven Avenue.

The roadway that accommodates bridge traffic coming from points north on Riverside Drive begins at a point on the west side of the Drive approximately opposite 169th Street. Traffic enters the roadway by a right turn from the Drive, follows the roadway as it curves north, passes over Riverside Drive by way of an arch opposite West 171st Street, and there joins the other branch of the "onbound" roadway previously described.

The roadway connection carrying "offbound" bridge traffic to Riverside Drive begins at Haven Avenue, at the end of the approach roadway that runs parallel to West 178th Street and extends in a southern direction, parallel with the "onbound" roadway to a point approximately opposite West 176th Street where the roadway forks. The traffic intending to travel southbound on Riverside Drive continues on a branch that crosses over the Drive on the lower arch, previously described, and thence southward through Fort Washington Park to a junction with Riverside Drive at a point approximately opposite West 168th Street. Traffic northbound takes the roadway which turns north at the fork and enters Riverside Drive at the foot of West 177th Street.

The "onbound" and "offbound" roadways comprising the connections with Riverside Drive are 30 ft wide to accommodate three lanes of traffic, except in the case of the "onbound" roadway south of 172d Street, which has a width of 35 ft to provide for parking facilities, and the "onbound" roadway for southbound bridge traffic on Riverside Drive which is 20 ft wide.

3.—Improvements Between Fort Washington Avenue and Broadway.—From the sub-surface plaza under Fort Washington Avenue, the final plan contemplates a ramp 40 ft wide, rising to a proposed plaza on the west side of Broadway (Fig. 4). From the same sub-surface plaza, "offbound" traffic may enter the vehicular tunnel under 178th Street. "Onbound" traffic will come to the plaza from a similar tunnel under 179th Street.

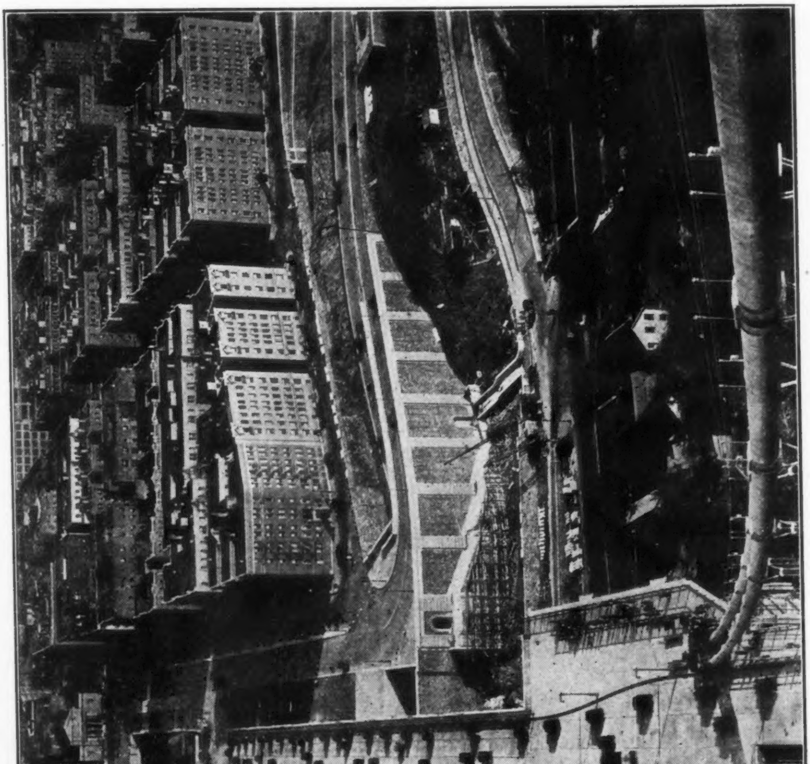


FIG. 6.—VIEW OF TRAFFIC OPERATION ON NEW YORK APPROACH, LOOKING EAST FROM TOWER.

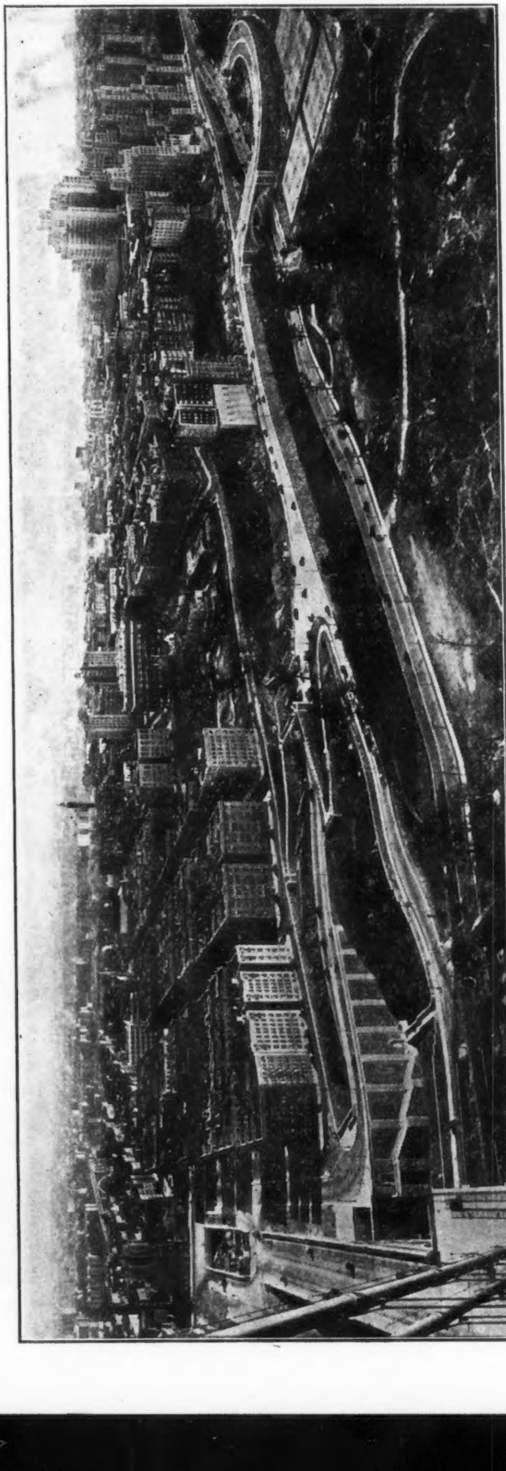


Fig. 7.—View of Approach Connections from Riverside Drive, George Washington Bridge.

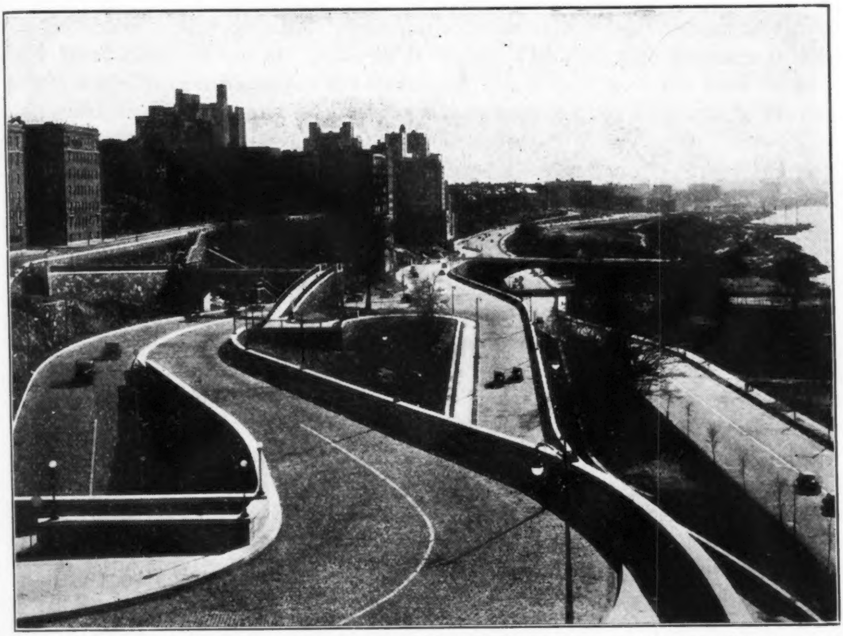


FIG. 8.—VIEW OF APPROACH CONNECTIONS TO RIVERSIDE DRIVE, LOOKING SOUTH FROM MAIN ROADWAY.

FIG. 9.—VIEW OF ARCH FOR MAIN RAMP OVER RIVERSIDE DRIVE, DURING CONSTRUCTION, APRIL 10, 1931.

In order to provide ample street capacity in this area, West 178th Street and West 179th Street are to be 80 ft wide. The plan also proposes in this block, a building suitable for the location. The structure would have arcaded sidewalks inside the building lines, making possible curb locations to provide roadways 63 ft wide.

Under initial conditions, only the tunnel in West 178th Street connects with the plaza under Fort Washington Avenue. West 178th Street is widened to 36 ft between curbs, but the buildings in the block between this street and West 179th Street west of Broadway are undisturbed.

4.—Tunnel Connections to Highbridge Park.—The vehicular tunnels under West 178th Street and West 179th Street, connecting with the sub-surface plaza at Fort Washington Avenue, have been mentioned previously. These tunnels, each two lanes in width, are designed to carry traffic to Amsterdam Avenue, the Harlem River Speedway, the Washington Bridge over the Harlem, and to any other crossing over the Harlem River that may be designed later. For the initial traffic condition the tunnel under West 178th Street only is being completed. A roadway from the portal in Highbridge Park just east of Amsterdam Avenue is being constructed by the Port Authority to connect with Amsterdam Avenue. The tunnel is to carry a single lane of traffic in each direction pending the completion of the second tunnel.

Approach Structures

One of the most interesting features of the New York approach is the arch that supports the main ramp over Riverside Drive. It has a clear span of 196 ft, a rise of 36 ft, and a clear height above the Drive of 66 ft at the crown. It is of the barrel type and is constructed of reinforced concrete. Fig. 9 is a view taken April 10, 1931, during the progress of construction. The arch is designed for a load of 235 000 lb per lin ft, such a high loading resulting largely from the provision made for a future lower deck capable of carrying four tracks of rapid transit.

The arch thrust of the west abutment is resisted by the anchorage of the main bridge. At the east abutment the thrust is transmitted into the rocky hillside through a comparatively small concrete block. The barrel design was chosen in preference to an arch with ribs because of the flexibility thus made possible in the location of railway tracks and because the barrel design made simplified construction possible. Under final conditions, the structure will be 145 ft wide, but initially the barrel width is limited to 65 ft. It varies in thickness from 7.5 ft at the springing line to 4 ft at the crown.

The superstructure above the arch barrel is a combination of concrete and steel. Concrete cross-walls spaced 24 ft apart were poured integrally with the arch barrel. These walls were brought to a level about 3 ft below the grade of the future lower-level ramp. They carry a steel superstructure of columns supporting plate girder cross-beams and suitable framing supporting a 9-in. concrete slab roadway reinforced with 6-in. bar trusses. The structure above the arch barrel is masked for the present with spandrel walls of mortar stucco on wire mesh. With the addition of the side roadways

and the widening of the arch to 145 ft, the superstructure will be faced with granite, as will the abutments and the remainder of the main approach ramp. A perspective view of the completed arch is shown on Fig. 10.

The main ramp contains no unusual type of construction, other than the arch, and consists of conventional braced steel framing on a concrete column substructure. The "onbound" roadway from Riverside Drive passes under the main ramp west of Haven Avenue and, at this point, the ramp roadway is carried by 92-in. plate girders having spans of approximately 66 ft.

The major part of the roadway system connecting with Riverside Drive is of the retaining-wall type, built into the hillside. Near the north end, however, a section of the "offbound" roadway (shown in Fig. 7), is designed as a rigid frame structure to resist the earth pressure. This design obviated the necessity of a gravity retaining wall which would have projected into the area desired for roadway use.

The lower arch over Riverside Drive (Fig. 11) is of reinforced concrete faced with stone; it has a span of 120 ft. The arch barrel is 75 ft wide. It is comparatively flat and utilizes a broken joint at the center of the span which relieves the arch from any floor action or wall action. The thrust of the arch on the east side is transmitted to outcropping rock by means of a small abutment. At the west end, it was necessary to construct a concrete abutment, 40 ft deep, 26 ft wide, and 80 ft long, because of the depth of rock satisfactory for foundation purposes.

The approach structure below the arch and on the west side of Riverside Drive is of fill within retaining walls. The retaining walls themselves were placed on fill and required continuous pouring of the footings in order to insure uniform settlement. Joints are provided at 50-ft intervals along the walls.

The tunnel in West 178th Street is built on the usual lines of subway construction. A cross-section, shown on Fig. 12, indicates that, in section, the tunnel is a concrete box about 40 ft wide and 20 ft high in outside dimensions. The steel framing is made up of bents, 5 ft center to center, which are encased in the concrete. The roadway is 22 ft in width with a clear height of 14 ft. Ducts for fresh air and for vitiated air are on one side, the fresh air duct being below the vitiated air duct. Transverse ventilation is provided, fresh air being admitted to the tunnel immediately above the curb on both sides of the roadway and the vitiated air removed through ports in the ceiling. A ventilation building for the tunnel in 178th Street, constructed for the initial traffic conditions, is situated about midway of the tunnel length.

The pavement on the main approach ramp is a reinforced concrete slab of 9-in. thickness. The approach roadways to Riverside Drive, where they parallel the main ramp, and down to the point of separation of the "onbound" roadways, are paved with granite block; south of that point they are of sheet asphalt on a concrete base, except on the curves where asphalt block paving has been used because of its non-skid properties. The tunnels in 178th and 179th Streets are to be paved with granite block.

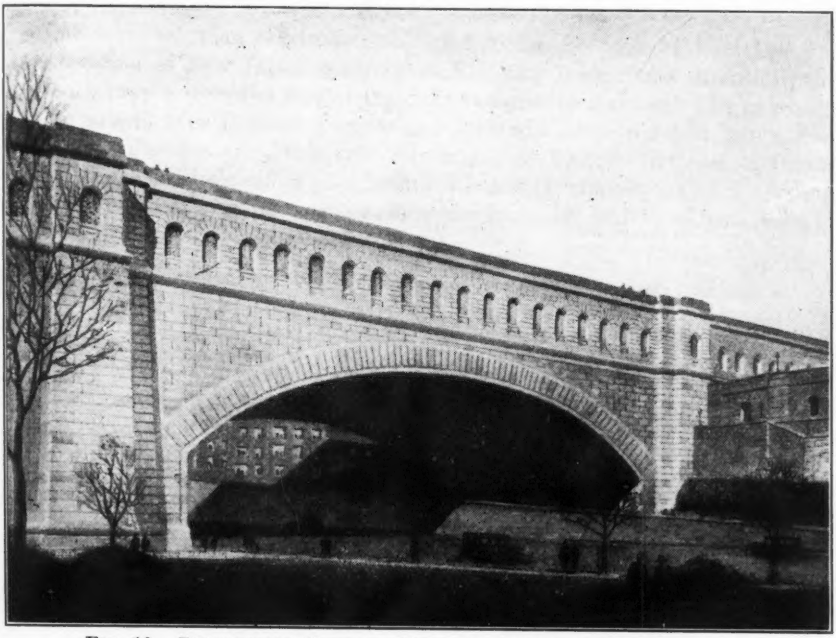


FIG. 10.—PERSPECTIVE VIEW OF MAIN ARCH OVER RIVERSIDE DRIVE.

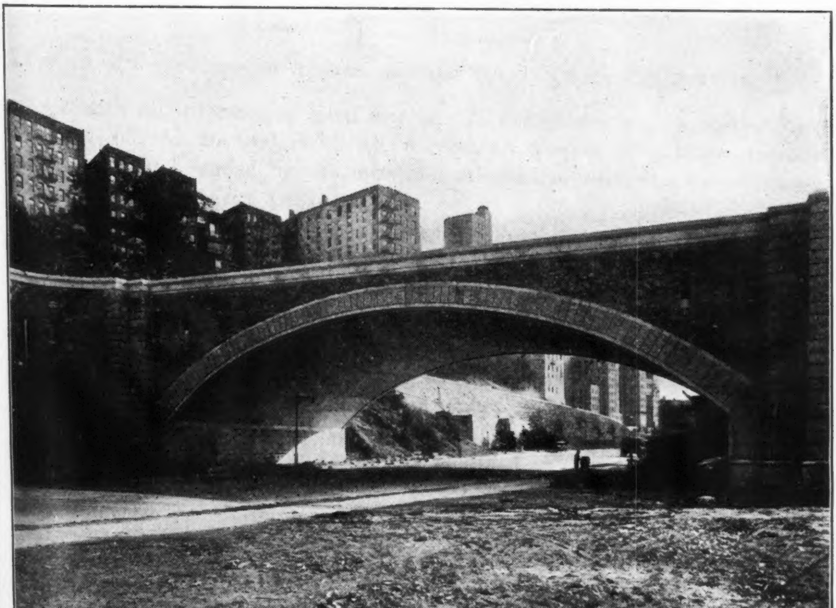


FIG. 11.—LOWER ARCH OVER RIVERSIDE DRIVE.

Development of the New Jersey Approach Plan

The approach plan originally submitted to the Borough of Fort Lee and the Governor of New Jersey is shown on Fig. 13. It embraced essentially the approach proper from the face of the cliffs to Lemoine Avenue. The approach was to extend over Hudson Terrace and then was to be widened into a spacious toll-collection area, this area continuing to Lemoine Avenue to terminate in a traffic circle. The plan included highway connections with Hudson Terrace and local marginal streets along the approach, which, in turn, were to

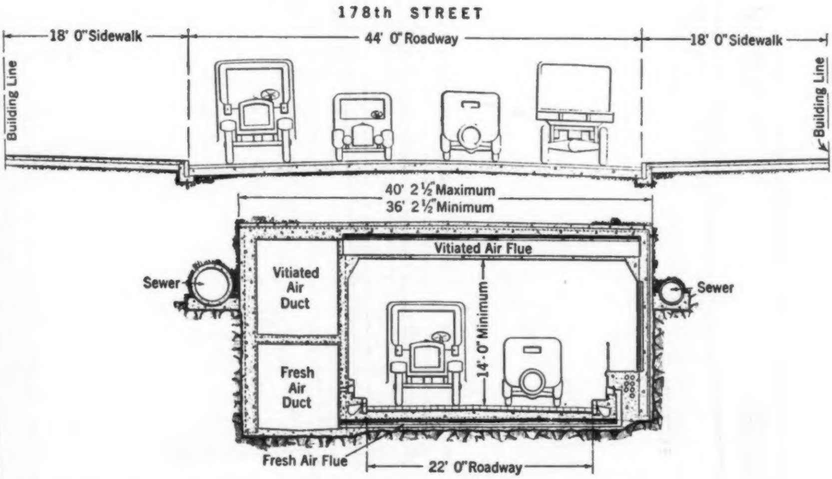


Fig. 12.—Cross-Section of Tunnel in West 178th Street, New York, N. Y.

connect with all intersecting local streets. In submitting the plan, the Port Authority offered to undertake all necessary changes and improvements within the area bounded by the marginal streets, imposing no expenditures on the Borough of Fort Lee.

The plan was approved by the Borough of Fort Lee and by Governor A. Harry Moore, of New Jersey, in the spring of 1927. At that time it was understood that Lemoine Avenue would be a State highway, improved and widened by the State, and would thus form, at least initially, the principal highway connecting with the bridge approach. It was also understood that, depending upon what the State Highway Commission might do in the way of improving Lemoine Avenue, or building new connecting highways, the plan approved by the Borough of Fort Lee and Governor Moore would have to be re-studied and probably modified in the vicinity of Lemoine Avenue.

An Engineering Committee composed of representatives of the State Highway Commission, Bergen County, the Borough of Fort Lee, and the Port Authority, undertook a thorough study of the connections. A large number of possibilites were studied, including those for grade separation at Lemoine Avenue, and for the extension of a wide boulevard west of the plaza

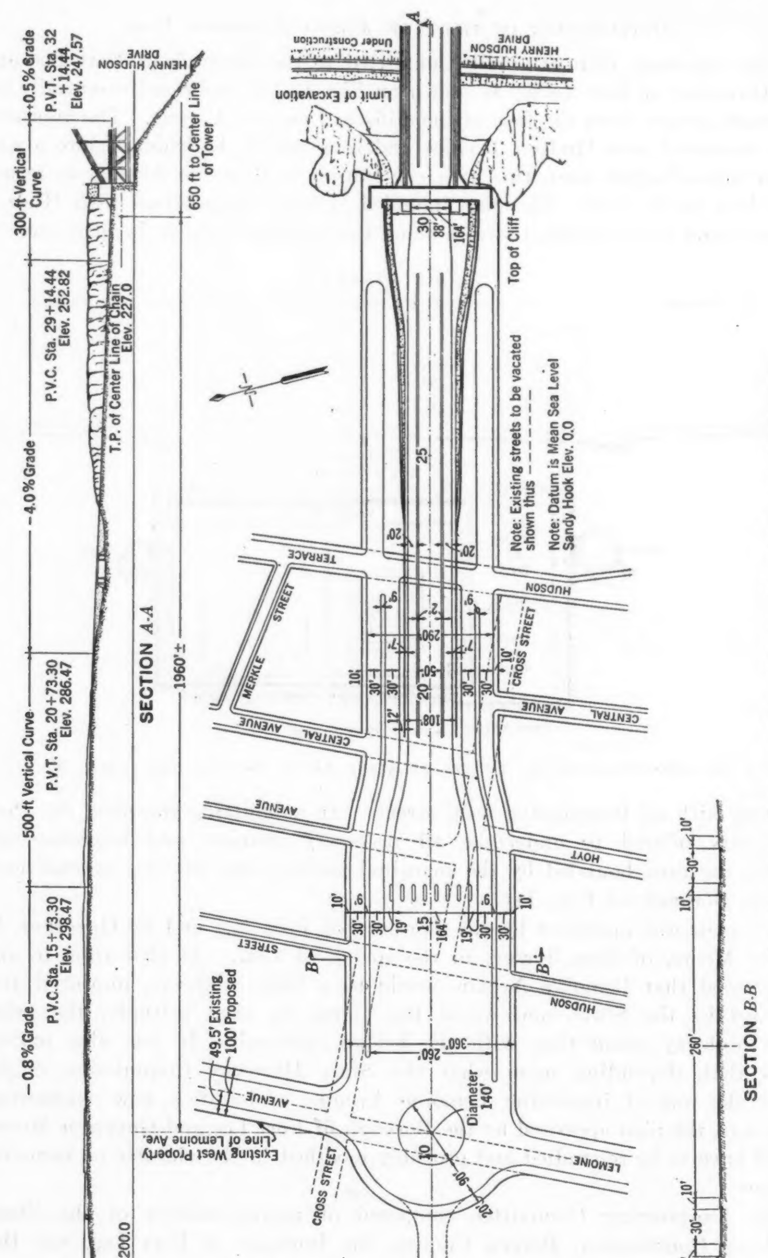


FIG. 13.—ORIGINAL PLAN OF NEW JERSEY APPROACH AS SUBMITTED.

to connect with arteries serving the Hackensack, Paterson, and Passaic areas. During the course of these studies the State Highway Commission developed a system of new and improved highways west of the bridge plaza designed to serve local, through, and bridge traffic.

In the early development of the plans it was intended that the work of the Port Authority should terminate at Lemoine Avenue and that the State should undertake the work to the west. However, as the negotiations between the State Highway Commission and the Port Authority progressed, the point of view was evolved that the proposed highway from Lemoine Avenue to the connection with State Highway Route No. 4, near Fletcher Avenue, a length of about 2 000 ft, should be considered at least partly as an essential feature of the bridge approach system, necessary to maintain traffic flow to and from the structure, and properly should be built by the Port Authority.

The plan of the Port Authority as approved by both the Borough of Fort Lee and Governor Moore, and used as a basis for financing, had provided for the termination of the approach at Lemoine Avenue. Thus, no funds were available to meet the cost of the extension, estimated at $2 500 000.

The Port Authority then proposed that it would undertake to build the extension when the bridge traffic should reach 10 000 00 vehicles per year and as funds should become available, or that the State proceed to build the extension as part of the proposed State Highway, in which event the Port Authority would agree to reimburse the State to the extent of $2 500 000 when the bridge traffic should reach 10 000 000 vehicles per year and as the funds should become available. The latter alternative was finally adopted.

On December 27, 1929, the Engineering Committee submitted a report recommending a plan, shown on Fig. 14, "considered to be best suited to all conditions." The report, which is signed by J. L. Bauer, M. Am. Soc. C. E., State Highway Engineer of the New Jersey State Highway Commission (successor to William G. Sloan, M. Am. Soc. C. E.), R. P. McClave, Engineer of Bergen County, S. Wood McClave, M. Am. Soc. C. E., Engineer of the Borough of Fort Lee, and O. H. Ammann, M. Am. Soc. C. E., Chief Engineer of the Port Authority, states, in part:

"Safe and expedient handling of traffic has been the paramount aim in developing the plan, but due consideration has also been given the aesthetic effect of the plan on the landscape. An effort has also been made to retain existing conditions and the disturbance of alignment and grades of existing highways has been minimized. All gradients on major routes have been held to a 4% maximum. Certain minor connections have a steeper gradient, but all are well within easy operating possibilities.

"The plan provides for adequate facilities not only for the initial four-lane traffic over the bridge, but for the final eight-lane capacity of the upper deck and, for additional lanes on the lower deck, if necessary. The plan has been developed so that future modifications can be consummated with little or no disturbance to traffic."

To facilitate the actual work of construction an arrangement was made whereby the work done by the State Highway Commission would extend to a line about 470 ft east of the west street line of Lemoine Avenue. However, since the Port Authority had taken into consideration in its original plan

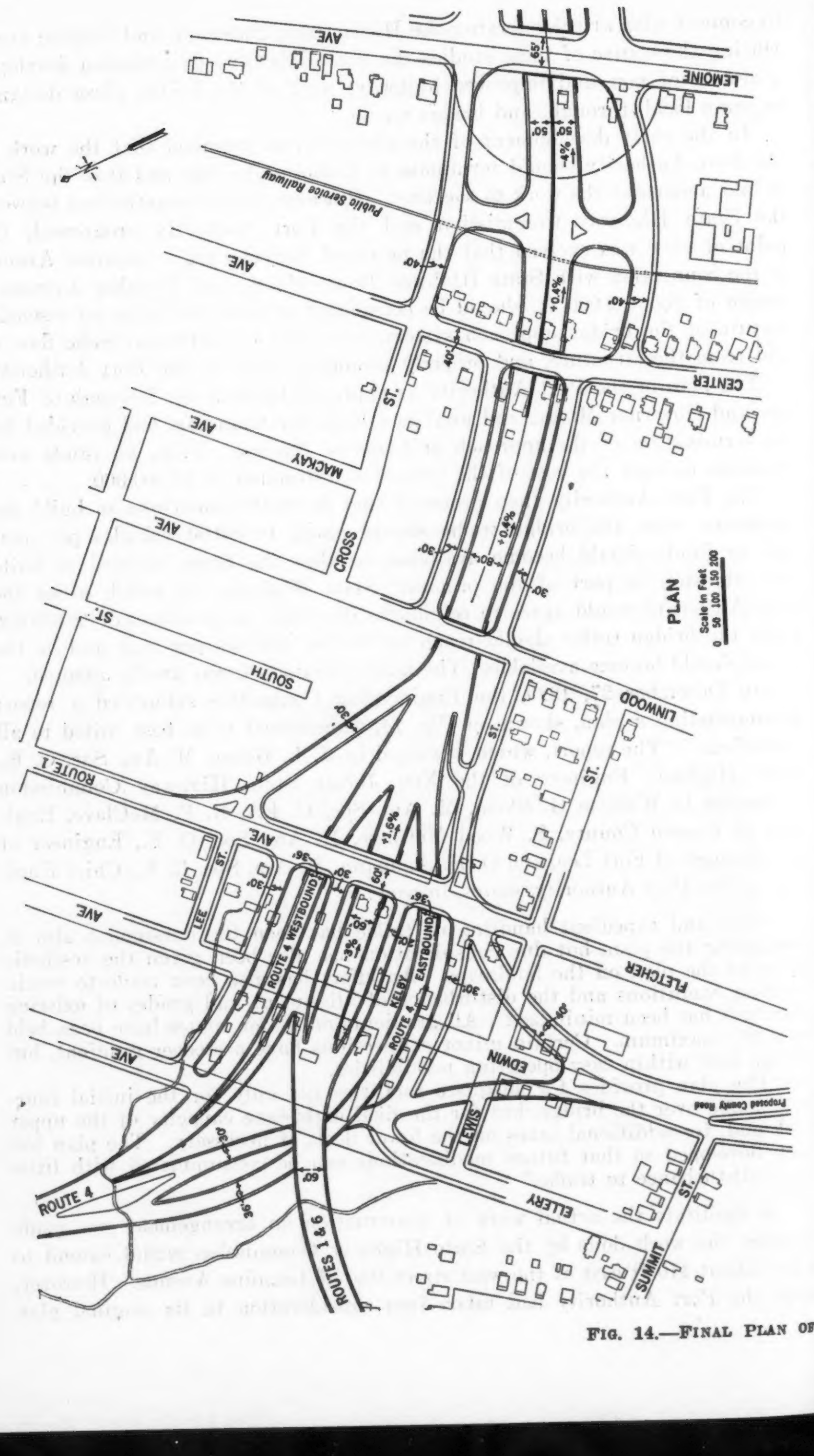
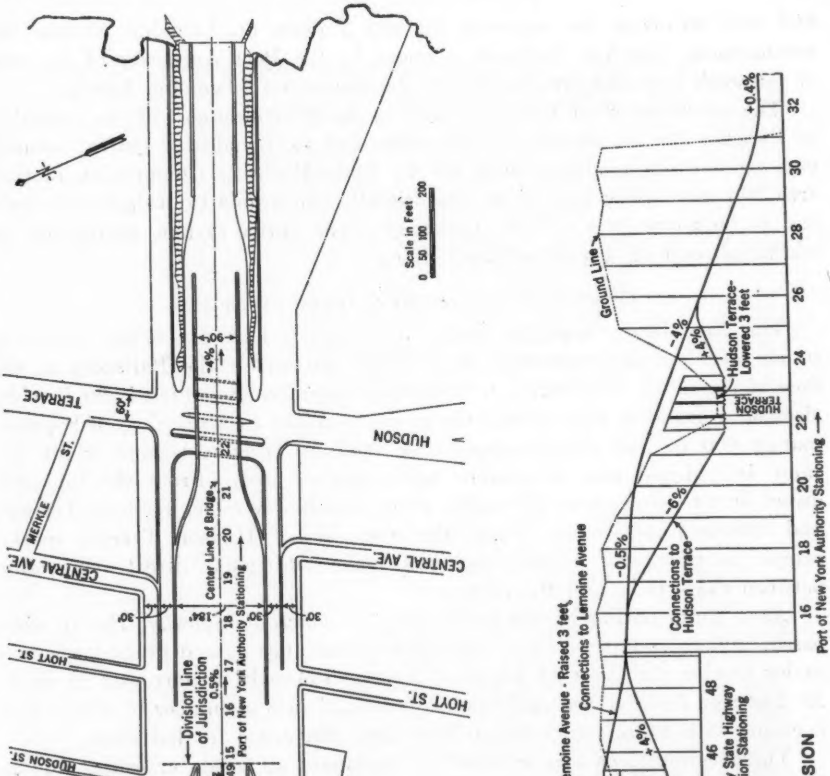


FIG. 14.—FINAL PLAN OF



NEW JERSEY APPROACH

and cost estimates the approach to, and a plaza at, Lemoine Avenue the arrangement therefore included payment by the Port Authority of the cost of the work from the division line to the west side of Lemoine Avenue.

The reference which has been made to the $2 500 000 cost of the extension to Fletcher Avenue should not be understood to include the entire expenditure on the new roadways built by the State Highway Commission in that area, but merely the part of the cost directly chargeable to bridge traffic only and to be borne by the Port Authority. The entire system aggregated an additional cost of several million dollars.

Description of the New Jersey Approach

The New Jersey approach (Fig. 15), where it joins the bridge proper is in an open cut approximately 50 ft below the top of the Palisades at the face of the cliff. Continuing westward, the main ramp, which is 90 ft wide, rises on a grade of 4%, attains the ground surface in a distance of approximately 600 ft, and thence passes over Hudson Terrace, beyond which the ramp is widened into a spacious toll-collection area. From the toll area direct street connections are made with Lemoine Avenue, Hudson Terrace, and intermediate streets. From the east side of Hudson Terrace special ramps to the main bridge approach make additional direct connection between that street and the bridge.

As a continuation of the main ramp, a concrete highway, 100 ft wide, passes under Lemoine Avenue and thence continues as a depressed roadway under Center Avenue and Linwood Avenue where it is narrowed to 90 ft. At Fletcher Avenue, the main ramp is divided into a number of direct connections with State routes which have been converged in that area.

These connections are designed to eliminate all grade crossings and to form a safe collecting and discharging agency for high-speed traffic. Cross-street connections within the Borough of Fort Lee are provided by means of ramps including a "clover leaf" layout between Lemoine Avenue and Center Avenue. Marginal streets between Center Avenue and Linwood Avenue connect with the "clover leaf" system.

The approach ramps, plaza, and connecting roadways have been built initially to the full width for the accommodation of ultimate bridge capacity, including any vehicular capacity which the lower deck may have if, and when, added at a future date.

In the design of the New Jersey approach access to the lower deck of the bridge has been considered only to such an extent as to insure that any construction incorporated in the original design shall in no way restrict connections to the lower deck at some future time.

New Jersey Approach Structures

The open cut through the Palisades was excavated as part of a contract which also included excavation of pits and tunnels for anchoring the cables. The cut in rock is 146 ft wide throughout its length and involved the excavation of approximately 200 000 cu yd of material. The only viaduct structure within the limits of construction by the Port Authority is the over-

FIG. 15.—AERIAL VIEW OF NEW JERSEY APPROACH, GEORGE WASHINGTON BRIDGE.

head crossing at Hudson Terrace where the approach ramp (90 ft. wide) is supported by a structure 135 ft long, consisting of three transverse steel bents of five columns each.

The excavation for the ramps to the west side of Hudson Terrace and for the widening of the plaza between Hudson Terrace and Lemoine Avenue required the removal of more than 75 000 cu yd of rock. The entire roadway ramp and plaza area is paved with concrete, with a slab thickness of 9 in. The construction west of the line of jurisdiction, separating the Port Authority and State Highway work, which has been mentioned previously, was done by the State Highway Commission.

Provision for Toll Collection

The facilities for the collection of tolls are at the Fort Lee end of the bridge in a spacious area west of Hudson Terrace and at the two side ramps leading eastward from Hudson Terrace to the bridge. A field office and garage are centrally situated just south of the toll area. The two are combined into a single structure of fireproof construction with an exterior finish of granite and sandstone rubble masonry.

The buildings for the collection of vehicular tolls extend across the plaza opposite the field office (Fig. 16). Seven pairs of booths serve fourteen lanes. A toll-house at either end of the line of toll booths serves an additional lane each, making a total of sixteen toll lanes. All the toll booths are connected by a continuous canopy which extends from toll-house to toll-house over the entire line. Provision for vehicular tolls is also made at the ramps leading eastward from Hudson Terrace to the main bridge ramp.

The toll-houses are two-story structures, 40 ft long and 12 ft 6 in. wide, of fireproof construction of granite exterior. These houses provide locker-rooms and showers for the collectors and also furnish space for an instrument room, a battery room, and rooms for tellers, collectors, and police sergeants. Heat is furnished for the toll booths and toll-houses from the field office.

The toll booths are also of substantial fireproof construction with frames of structural steel. These booths are plastered inside and are finished on the outside with sheet aluminum for architectural effect. Toll booths serving the ramps leading eastward from Hudson Terrace are, in general design, similar to the main group. A single pair of booths accommodates two lanes in each ramp. The field office and garage, toll-houses, and toll booths have been treated architecturally to conform to the general dignity of the structure.

Plaza Illumination

A special condition was encountered at the New Jersey toll-collection area where a space approximately 450 ft long by 250 ft wide had to be illuminated adequately without offering obstruction of any sort to vehicular travel. To meet this condition, four flood-light towers are placed near the corners of the area. Two of these towers are approximately 185 ft east of the line of the toll booths and two are approximately 280 ft west of the toll buildings in the area.

These towers are equipped with batteries of flood lights that illuminate the plaza brilliantly from a height of about 100 ft. The towers are of simple structural steel construction mounted on granite bases. They are painted to harmonize with the general bridge structure.

Vehicular Toll Collection Equipment

Automatic registering equipment has been installed at the toll booths. This equipment consists of four main elements:

(1) Fare registers for recording the amount of tolls.

(2) Tariff signs for indicating the classification of vehicles as registered by the toll collectors.

(3) Central office indicator boards for remote checking of vehicle classifications as registered by the collectors.

(4) Vehicular treadles for counting and recording the axles passing through each toll lane.

The tariff signs, indicator boards, and fare registers are inter-connected electrically while the vehicular treadles are electrically independent. These devices function as follows: When a vehicle enters a toll lane the operator stops momentarily at the booth to pay his toll. The collector receives the fare, makes change as necessary, and permanently records the transaction in printed form on the register according to classification of vehicle and amount paid. He accomplishes this by two simple motions of an actuating lever. The manipulation of the fare register also causes illuminated figures representing the classification of the vehicle to appear on a pair of overhead tariff signs erected at the entrance and exit of the toll lane (see Fig. 17). These signs are placed and arranged so that they are visible from practically all points on the plaza, such an arrangement giving the officers, or others in charge of the collection of tolls, a ready means of checking.

The officer in charge has a further check upon the collection of tolls, because when the collector registers a vehicle, the classification number (the same number as indicated on the overhead signs) is shown on one or more panel boards located centrally in near-by toll-houses, or in the field office near windows overlooking the plaza. These boards are designed to permit simultaneous checking of several collectors. They are distinguished from the tariff signs by the fact that, unlike the latter, they operate only at the will of the checker who throws a small switch for each lane that he wishes to observe.

As far as the operator of the vehicle is concerned, the toll collection operation is complete with the payment and the registration of the fare. No tickets are issued to be collected later. Instead, as the vehicle leaves the toll lane, it passes over a treadle in the pavement. The treadle operates electrically, counts each of the axles passing over it, and records these impulses by a mechanical printing operation in the fare register on the same sheet of paper with the classification count of vehicles. As vehicles are classified according to number of axles as well as to type and capacity, an exact check between the collector's record and the treadle count is possible.

FIG. 16.—TOLL COLLECTION FACILITIES, NEW JERSEY APPROACH, GEORGE WASHINGTON BRIDGE.

FIG. 17.—CLOSE-UP VIEW OF TOLL BOOTHS.

All the aforementioned indicating and registering equipment has been designed specially for installation on this and other Port Authority toll projects and much of it represents pioneer work in this field.

Pedestrian Toll-Collection Facilities

Pedestrian tolls are collected on the approaches at either end of the bridge. Pedestrians pass through turnstiles housed in a suitable structure at the New Jersey anchorage. The building furnishes space for the cashier's office and for police purposes.

Similar facilities are placed at the east end of the arch over Riverside Drive where arrangements for pedestrian accommodation require passage through the interior of the arch structure to a stairway leading to the sidewalks at the anchorage.

Acknowledgments

It has been intended in this paper to indicate clearly the close co-operation with the various Municipal, County, and State agencies that was sought and received by the Port Authority.

In the development of the New York approach plan and in facilitation of construction operations the co-operation of Arthur S. Tuttle, M. Am. Soc. C. E., Consulting Engineer of the Board of Estimate and Apportionment of the City of New York, Clifford M. Pinckney, M. Am. Soc. C. E., Chief Engineer of the Borough of Manhattan, Robert Ridgway, Past-President, Am. Soc. C. E., Chief Engineer of the Board of Transportation, Edward A. Byrne, M. Am. Soc. C. E., Chief Engineer of the Department of Plant and Structures, and the late John R. Slattery, M. Am. Soc. C. E., Deputy Chief Engineer of the Board of Transportation, was very helpful. The following members of the Department of Water Supply, Gas, and Electricity—the Hon. John J. Dietz, Commissioner, Nicholas J. Kelly, Chief Engineer of Light and Power, William W. Brush, M. Am. Soc. C. E., Chief Engineer, Bureau of Water Supply—and numerous other officials of New York City Departments, have also rendered their hearty co-operation.

William G. Sloan, M. Am. Soc. C. E., former State Highway Engineer, and, later, J. L. Bauer, M. Am. Soc. C. E., the present State Highway Engineer of the New Jersey State Highway Commission, Hugh A. Kelly, Assoc. M. Am. Soc. C. E., R. P. McClave, County Engineer of Bergen County, and S. Wood McClave, M. Am. Soc. C. E., Engineer, Borough of Fort Lee, and other officials have co-operated with the Port Authority in the development and construction of the New Jersey approach.

Officials and members of the Public Utility Companies in both the State of New York and in the State of New Jersey, have rendered valuable assistance, as have also various civic bodies.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

PUBLIC SUPERVISION OF DAMS
A SYMPOSIUM

Discussion

BY MESSRS. A. H. MARKWART, AND M. C. HINDERLIDER

A. H. MARKWART,[28] M. AM. SOC. C. E. (by letter).[28a]—Consideration by the Society of the subject of Public Supervision of Dams has invoked liberal discussion from its members. This discussion, which represents the opinions of engineers interested or engaged in the design and construction of dams, should be of much value in forming a basis for the preparation of future legislation.

The writer's opinions, as expressed in his paper, were briefly as follows:

(1) While it is to be regretted that resort to Government is necessary to effect correction, State supervision of dams is apparently to be had.

(2) Supervision should be conducted through the medium of existing governmental machinery—the State Engineer, or equivalent—for administration, and the Courts for enforcement.

(3) Administrative duties should be vested in one office, and the statute should be confined to basic principles, leaving the details to be covered by regulations and rules.

(4) Supervision should be limited to the approval of the location, design, construction, and maintenance of the dam in so far as safety alone is concerned.

(5) No dams should be exempt from supervision because of ownership.

NOTE.—The Symposium which includes the paper by A. H. Markwart, M. Am. Soc. C. E., presented at the meeting of the Power Division, New York, N. Y., January 16, 1930, and the paper by M. C. Hinderlider, M. Am. Soc. C. E., presented at the Technical Session, Sacramento, Calif., April 23, 1930, respectively, was published in January, 1932, *Proceedings*. Discussion of the Symposium has appeared in *Proceedings* as follows: March, 1932, by H. deB. Parsons, M. Am. Soc. C. E.; April, 1932, by Messrs. William P. Creager, M. M. O'Shaughnessy, N. A. Eckart, R. C. Johnson, F. W. Hanna, and Joel D. Justin; May, 1932, by Messrs. I. C. Steele and Walter Dreyer, Fred A. Noetzli, George N. Carter, George W. Hawley, and H. K. Barrows; and August, 1932, by Messrs. Harry W. Dennis, N. Kelen, Ford Kurtz, and William W. Tefft.

[28] Vice-Pres. in Chg. of Eng., Pacific Gas & Elec. Co., San Francisco, Calif.

[28a] Received by the Secretary December 19, 1932.

(6) Small dams that cannot become a menace should be exempt from supervision.

(7) The burden of the cost of supervision should be divided between the owner and the State.

(8) Provision should be made for appeal either to the Courts, or to a board of arbitrators, in the event that the owner is reluctant to follow the decision of the supervisory official. Such appeals should be expedited to avoid delay.

(9) Legislation should provide for the inspection of existing dams and should confer power on the supervisory agency to order necessary remedial measures.

(10) Routine inspection should be made of all important dams.

(11) It is suggested that a "minimum requirement" code be developed, although this may prove to be difficult or impracticable.

It is interesting to comment on the manner in which the foregoing points impressed the members who discussed the paper. In general, State supervision was considered desirable, although Mr. O'Shaughnessy and the late Mr. Tefft commented adversely on this point. Mr. Hanna believes that State supervision will not meet all problems, and suggests some method of National control for intra-State projects.

In the matter of supervisory personnel, Mr. Parsons suggested for important structures the appointment of a supervisory board, to approve or reject the design and foundations of the dam. In the event of dispute, an appeal would be made directly to the highest Court having jurisdiction. Technical matters under appeal would be referred to a new board appointed by the Court. Mr. Kurtz suggested the appointment by the Governor of a permanent State Commission of not less than six competent engineers, and the vesting of all authority in the Commission, which would use existing agencies for carrying on routine duties.

In the matter of appeal, most of the comment was in favor of an appeal to a board of arbitration rather than to the Courts, in order to minimize delays.

The subject of the practicability and desirability of a code was discussed with considerable difference of opinion. Several thought a code desirable and practicable; others thought it desirable, but either difficult or impracticable of formulation at this time; while still others considered it undesirable in any event. If the consensus of opinion is correctly interpreted, it seems that some kind of minimum requirement code, general in its terms, but not incorporated in the law, is considered desirable, and will ultimately be developed.

In conclusion, the writer desires to reiterate that, in his opinion, the real values of State supervision lie in the independent review of the original design and construction, so the problem is not entrusted entirely to one man, and in the continuity of technical inspection and control long after the interest of the designers and constructors in the structure have terminated.

M. C. HINDERLIDER,[29] M. AM. SOC. C. E. (by letter).[29a]—As one of the authors of this Symposium, the writer desires to express his deep appreciation of the interest shown by the profession in this important subject, as manifested by the numerous interesting and valuable discussions. Such discussions have brought out many valuable points pertinent to the subject, and not covered by the authors. With but one or two exceptions, they disclose a unanimity of thought as regards the necessity for public supervision of dams and the essentials to be incorporated in such supervision.

Regulatory laws covering public supervision of dams, as suggested by Mr. Parsons, would be too unwieldly and unnecessarily burdensome, would result in great delay in initiating construction, and would be impracticable of efficient administration. No distinction should be drawn as between high dams impounding large volumes of water, and those of nominal heights impounding comparatively small quantities of water. Experience, particularly in the Western States, shows that even nominal volumes of water, when suddenly released, create momentary discharges far in excess of the capacity of the natural channels to carry them, and hence cause great devastation to areas lying below such structures. A dam of only nominal height may create a greater menace, under certain conditions, than one of much greater height and impounding capacity.

Since 1925 the only two dam failures in Colorado (where there are about 1 200 dams under public supervision) have been small earth structures, each about 17 ft in height, impounding less than 100 acre-ft; yet in each case a life was lost and considerable property damage resulted. Hence, there should be no distinction drawn as regards the degree of public supervision of dams which impound even inconsiderable quantities of water, since such impoundment may be located immediately above habitations, arteries of transportation, and valuable property rights. In this connection, the writer agrees most heartily with the views expressed by Mr. Creager.

The writer is in accord with the objections of Mr. Eckart concerning the danger of political influence entering into State supervision of dams. As in all functions of public officials vested with police authority, the opportunity is present for the transgression of the rights of the public. Apparently, the only remedy for this danger is reliance upon Court review of the acts of the administrative official, which right is generally conceded, whether or not specifically provided by the laws clothing the administrative official with his authority. Undue restriction upon the authority of the administrative official only tends to nullify the objective sought, that is, the protection of the public as to life and property, which may be placed in jeopardy by the erection of works of the nature under consideration. Where the public official charged with the supervision of dams is either incapable or over-conservative in the interests of the public, to the detriment of the owner of a dam, and where the situation is sufficiently justifiable, the owner can always secure redress through the Courts.

---

[29] State Engr. of Colorado, Denver, Colo.

[29a] Received by the Secretary December 23, 1932.

The suggestions of Mr. Hanna respecting public supervision of dams that may be constructed on an interstate or international stream, are very pertinent, and are food for much careful thought, especially since in the future the construction of dams of material size will be, in many instances, on stream systems, interstate or international in character. This problem is twofold: First, in its effect upon the relative rights of the States or nations, with respect to the utilization of the waters impounded by such structures; and, second, as regards safety of life and property in an adjoining State below such structures. In the arid regions of the West, the actual consumptive use of the natural water supplies is essential to existence and habitation, while the use of this all-important resource for domestic, sanitary, and manufacturing purposes, is equally vital to the inhabitants of the humid regions. The writer believes firmly that, prior to the construction of any regulatory dam of any considerable size, on any interstate stream, a determination should first be made of the relative interests of each State affected, and proper provisions made for safeguarding such interests and rights, both in the actual uses of the waters thereof, and for the protection of life and property below such structure. Adjustments as regards water utilization may best be effectuated through interstate agreements or compacts. Failure to attain this end results eventually in suits in equity between the States, which can only be decided by the Supreme Court of the United States. Such procedure results in long drawn out litigation of a most expensive nature, and such decisions, of course, cannot be mutually satisfactory. Questions of design and safety may best be determined by joint high commissions appointed by the sovereignties whose rights are affected.

Discussions of the new law of California, covering the construction and supervision of dams, by Messrs. Steele, Dreyer, and Hawley, are most interesting and valuable, especially to those in other States, where more rigorous laws for public supervision of dams are contemplated.

Plans and specifications for two of the largest dams ever contemplated (San Gabriel and Pine Canyon), were presented to the State Engineer of California immediately following the adoption of the present California law, and, in this connection, the writer had the privilege of acting as a member of a commission appointed by the State Engineer of California to pass upon the adequacy of the plans submitted and the suitability of the sites for such dams. As a result, the functioning of this outstanding attempt by the people to apply complete supervision to works of this nature has been watched with interest.

Dr. Kelen's discussion of the laws governing public supervision of dams in Germany is also most interesting and profitable, and indicates rather definitely that a code embodying the essentials for safety, is workable. The new regulations covering public supervision of dams in Prussia, entitled "Instructions for the Design, Construction, and Operation of Dams," is a most valuable addition to the common fund of information on this important subject. It is interesting to note Dr. Kelen's comments that thus far "German engineers are not quite convinced that dams constructed by the hydraulic

method are sufficiently safe." The writer quite agrees that, in the absence of the application of proper engineering knowledge and experimental data pertinent to such design, such types of dams are pregnant with danger. Nevertheless, Terrace Dam, across the Alamosa River, in Colorado, has withstood the test of time most successfully. This dam is about 175 ft high, and all but the top 32 ft was constructed by the hydraulic method. The materials used, however, lent themselves admirably to such type of construction. Such requirements, of necessity, limit the presence of clayey materials to the minimum necessary for providing the requisite degree of imperviousness.

The completion of the new Eleven-Mile Canyon Dam by the City of Denver may be mentioned as an instance of effective co-operation between the owners of such a structure and the public officials charged with the duty of supervision over both design and construction methods. This dam is of the semi-gravity arched type, of a maximum height of about 150 ft., constructed at a cost of about $1 250 000. This is one of the best constructed and most beautiful structures of the kind to be found in America. The plans were prepared by the engineers of the Board of Water Commissioners of the City of Denver, and submitted to the State Engineer of Colorado, for approval. They were then referred to the Chief Designing Engineer of the U. S. Bureau of Reclamation for consideration and criticism. Following such studies, the writer felt that a design involving a length of radius between the two limits suggested by the City and the Government Engineers would best fit the topographic conditions at the dam site, and the final draft of the plans was predicated upon this determination.

Following the preliminary stages of excavation of the foundations for the dam, experienced geologists were retained to pass upon the geological features of the foundations. Fortunately, the foundations are in granite of excellent character. The only undesirable feature discovered was a fracture extending diagonally across the right abutment near the top of the dam. Prior to the placing of concrete, the writer made a critical examination of every foot of the foundation area and designated points where pressure grouting was to be applied. All materials that entered into the construction of the dam were manufactured from the granite excavated from the spillway section. The City provided a well-equipped laboratory on the ground for testing all aggregates and concrete. Tests were made under the immediate supervision of a representative of the State Engineer, who remained constantly on the work and sampled every day's pour of concrete, and also the pressure grouting in the foundations. Records of each day's pour of concrete, and the gradation and other characteristics of the concrete aggregate, were compiled by the State Inspector, and they constitute a complete and valuable record of the history of the construction of this dam. Throughout the period of construction the finest spirit of co-operation prevailed between the City's engineers and inspectors, the contractor, and the representatives of the State Engineer.

During the same time the Penrose-Rosemont Dam was also under construction in Colorado. The designing engineer for this rock-fill dam kept constantly in touch with the work. Prior to the commencement of construction he conferred many times with the State Engineer in the preparation of

his designs, and during the construction period, retained on the work an engineer qualified through experience to supervise the work carefully. As in the case of the Eleven-Mile Canyon Dam, competent geologists were retained to pass upon the foundation conditions, which were much more complicated than at the Eleven-Mile Canyon Dam, but which were treated successfully through deep pressure grouting and drainage. This latter dam involves some unique features, in that the water face is rendered impervious by means of a steel diaphragm laid over the up-stream face of rubble masonry, and electrically welded in place. Contacts between the diaphragm and the foundations consist of concrete emplacements overlying sound but fractured bed-rock. The seams and rock fractures were sealed later by heavy pressure grouting, through two lines of holes drilled at suitable depths through the concrete toe-wall.

No little part of the discussions of the paper related to a proposed code to cover the design and construction of dams, the opinions being about equally divided concerning the desirability of such a code.

It was not the writer's thought that a general code covering the entire field of dam design and construction could be devised, which would be applicable to all conditions that might arise, since it is realized that each dam and the controlling conditions appertaining thereto are problems within themselves, and accordingly must be treated as such. It was the writer's feeling, however, that a code covering the fundamentals that generally govern the design for any type of dam, might well be adopted for the guidance of State officials charged with the duty of passing upon such matters. These fundamentals would include such items as perviousness and permanency of foundation and structural materials, limitations on unit stresses and pressures, factors covering uplift and drainage, temperature changes, effect of earth movements, or temblors, etc. It is realized that a public official charged with the duties of passing upon the adequacy of design and foundation conditions, may not be fully versed in such matters, and, as a result, he might defer to the presumably superior knowledge and experience of the designing engineer. If, however, he was required by law to take into consideration all the fundamentals appertaining to the final safety of the structure, it would, to this extent, eliminate many probabilities of faulty design and construction, and to that extent would be in the interests of safety to the public. There are innumerable instances in which the magnitude of the proposed work does not justify the employment of expert advice, other than that furnished by the approving State official. Where his duties are not specifically enumerated in sufficient detail, the State official is prone to depend upon his own experience and judgment, which may not be justifiable. As a result of his official approval under such conditions, the best interests of the public, as well as of the owner of the structure, are not properly conserved. To this end the writer believes that a limited code, involving the fundamentals of dam design and foundation requirements, could be incorporated in a code with great advantage to the public, even if, with the advancement of the art of engineering, such code would probably have to be amended from time to time.

Acknowledgment.—The writer also desires to acknowledge the use of the report of "Hydraulic Power Committee, Engineering Section, J. E. Stewart, M. Am. Soc. C. E., Chairman, Presented at the Twenty-third Annual Convention of the Pennsylvania Electric Association," and Mr. Stewart's tabulation of dam failures, and to extend to Mr. Stewart full credit for the use of such information in the writer's compilation of the failures of 305 dams in the United States and foreign countries. In the bibliography of such failures noted on the tabulated list incorporated in the writer's paper, due credit for the foregoing reference was, unfortunately, omitted in the writer's compilation of the list of failures.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

STRESSES IN INCLINED ARCHES OF MULTIPLE-ARCH DAMS

Discussion

BY MESSRS. GEORGE E. GOODALL AND IVAN M. NELIDOV

GEORGE E. GOODALL[23] AND IVAN M. NELIDOV,[24] ASSOC. MEMBERS, AM. SOC. C. E. (by letter).[24a]—One of the discussers, Mr. Cochrane, presents formulas for moments, thrusts, and shears due to water load, dead load, and temperature changes in the inclined arches of multiple-arch dams. Although these formulas were derived by a different method and appear to be entirely different from those of the writers, they give the same numerical results. By substituting the quantities given in his Equations (76) and (77) into the equations for the crown thrust, H_0, given in Table 3, and multiplying the numerator and denominator of the resulting expressions by $\frac{k^2}{r^2}$, the equations for the crown thrust may be shown to be identical to those of the writers. The close agreement of the numerical results is thus accounted for and affords a valuable check on the accuracy of the work.

Mr. Cochrane expresses doubt as to the applicability of the "two-hinged" arch to multiple-arch dams. The writers undertook the derivation of the "hinged" arch formulas in order to analyze two completed dams of this type. In the case of these dams, the ends of the arches were rounded, and every effort was made in construction to cause the arches to act as if they were hinged at the ends. It is interesting, in this connection, to note that a more beneficial stress distribution is obtained in these "hinged" multiple-arch dams than in similar structures having arches with fixed ends.

It has been stated by Mr. Cochrane that the shears are always small and, ordinarily, need not be computed. This will usually be true in the more

NOTE.—The paper by George E. Goodall and Ivan M. Nelidov, Associate Members, Am. Soc. C. E., was published in March, 1932, *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: October, 1932, by Messrs. Victor H. Cochrane, Paul Bauman, Lars R. Jorgensen, Fred A. Noetzli, and W. J. Stich.

[23] Senior Engr., Hydr. Structure Design, Div. of Water Resources, Dept. of Public Works, Sacramento, Calif.

[24] Senior Engr. of Hydr. Structure Design, State Dept. of Public Works, Sacramento, Calif.

[24a] Received by the Secretary December 20, 1932.

recent designs, but in some of the older designs, shearing stresses of 90 lb per sq in. have been found. Unit shearing stresses of this magnitude would not be objectionable were it not for the fact that they were accompanied by tensile stresses at the same section. In the later designs of multiple-arch dams with their greater buttress spacing, larger central angles, and smaller ratio of thickness to mean radius, the shearing stresses are less important.

In the numerical example given by the writers, the temperature drop was chosen as 20° arbitrarily to demonstrate the method. It is well known that temperature changes greatly in excess of this may be expected. That the effect of shrinkage is of great importance must be admitted, but unfortunately the data on which to base any computations are very meager and until more is known of shrinkage quantitatively, it would seem to be wiser to take care of it by using a larger factor of safety, or by using a greater drop in temperature, as Mr. Cochrane suggests.

If it is desired to investigate the effect of a difference in temperature between the extrados and the intrados, the crown thrust due to such difference becomes:

$$P_0 = \left[\Delta x - \theta r_n \left(\frac{1 - \sin \phi_1}{\phi_1}\right)\right] \frac{Et}{r} \frac{2k^3}{D'_s r^3} \qquad (84)$$

in which, $\frac{2k^3}{D'_s r^3}$ is obtained by dividing the value of $\frac{2\phi_1 \sin \phi_1 t^2}{12 D_s r^2}$ (taken from Fig. 11) by $\sin \phi_1$. In Equation (84) the other terms are obtained from the following:

$$\Delta x = \frac{r}{t} (u_e r_e - u_i r_i)(\phi_1 - \sin \phi_1) + u r \sin \phi_1 \qquad (85)$$

and,

$$\theta = (u_e r_e - u_i r_i) \frac{\phi_1}{t} \qquad (86)$$

in which, u_e = unit deformation of extrados; u = unit deformation of center line; and u_i = unit deformation of intrados.

The thrust at any point is:

$$P = P_0 \cos \phi \qquad (87)$$

the moment at any point is:

$$M = P_0 r \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi\right) \qquad (88)$$

and the shear at any point is:

$$S = - P_0 \sin \phi \qquad (89)$$

Mr. Cochrane, as well as Mr. Jorgensen, suggests the advisability of increasing the thickness of the arches at the abutments. This has been recognized in some of the more recent designs. The approximate method used by the writers in their own work is that suggested by Mr. Cochrane. If the amount of increase in abutment thickness is great when compared to the crown thickness, it would be better to make the more exact calculations by

some method that considers the variable thickness of the arch ring, such as the formulas of von Müller-Breslau.[25]

Mr. Jorgensen questions the writers' statement that reinforcing steel is required in both faces of the arch. He admits the necessity of this when the abutment section is considered. In the arches of multiple-arch dams having moderately large central angles, the variable water load causes large positive moments at the crown. If the water surface is drawn down to the arch under consideration, it only requires a small rise in temperature to induce tensile stresses at the extrados of the crown. Due to the thin sections used in the arches, the rise of temperature in the concrete due to setting is quite small, and its subsequent dissipation would be equivalent to only a slight drop in temperature which would be insufficient to eliminate the tensile stresses.

The tendency to an increase in the central angle of the arches in recent designs is pointed out by Mr. Noetzli. Any one who has analyzed even a few typical multiple-arch dams will agree with Mr. Noetzli as to the advisability of this course. With better methods of designing buttresses and the application of the elastic theory to the analysis of the arches already an accomplished fact, the writers can only concur in Mr. Noetzli's assertion that "further improvements in the art are to be looked for rather from the point of view of construction than from that of design."

Mr. Stich presents calculations which demonstrate that even if the elastic theory is applied to the determination of stresses due to uniform water load, the effect of the weight of arch and variable water load must be considered. He, like others, points to the advisability of increasing the thickness of the arches at the abutments.

The writers made the statement in the "Introduction" to their paper that Dr. Kelen did not present curves from which the bending moments, abutment thrust, and shear could be obtained directly. Mr. Bauman asserts that this statement is misleading and that Dr. Kelen presented curves from which the unit stresses at the abutment may be obtained directly. It is true that these stress curves are included in Dr. Kelen's book, but they are for plain concrete. Inasmuch as the arches of all multiple-arch dams on which the writers have been able to gather data are reinforced, it was considered that if the curves were to cover the widest field of usefulness they should express the moments, thrusts, and shears from which the unit stresses for any percentage of reinforcement could be computed.

As Mr. Bauman states, the circular arch is not the most advantageous for the inclined arches of multiple-arch dams. However, the writers were presented with the problem of determining the stresses in more than twenty multiple-arch dams already in existence and of these structures, all had circular arches, except two.

According to Mr. Bauman, "the introduction of the influence of shear is a refinement, but is more of academic than of practical value * * *." In support of this contention he introduces stresses for Elevation 140 of the writers' example computed both with and without the influence of the shear-

[25] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1296-1304.

ing forces. The arch at this elevation has a thickness of only 2.58 ft and $\frac{t}{r}$ is 0.100. For such a value of $\frac{t}{r}$, the influence of the shearing forces is small, but had he chosen the arch at Elevation 40 where the thickness was 5.50 ft and $\frac{t}{r}$ was equal to 0.202, he would have found the effect of the shearing forces to be considerably greater.

To demonstrate the effect of shear on this arch the stresses have been computed for the various systems of loads with the effect of shear included and also with that influence neglected. When the elastic work of shear is omitted, the formulas for moments, thrusts, and shears are the same as those given in the writers' paper except that all terms containing the factor, 2.88, are omitted. For the arch at Elevation 40, it has been found from these calculations that omitting the effect of shear causes an error in the stresses due to uniform water load of 21% at the extrados at the abutment. For variable water load the maximum error due to the neglect of the work done by shear is at the extrados at the crown and amounts to 22 per cent. In the analysis for dead load, the maximum error occurs at the intrados at the crown and is 10 per cent. For the combined stresses due to the total load, the error due to omitting the effect of shear from the calculations is just 20% at the extrados at the abutment. The writers would consider the inclusion of a factor that affects the final result by 20% as having practical as well as academic value.

The writers concur in Mr. Bauman's statement that uniform strength throughout the arch ring is desirable but, unfortunately, this desirable condition is not easy of accomplishment.

Mr. Bauman presents a comparison between the stresses in a "pressure line arch" and those of a circular arch dam of approximately similar dimensions. In computing the stresses in the "pressure line arch" he presents Equation (82) for determining the elastic thrust, H_e, due to live load and a change in temperature. That any results at all comparable with those attained by the writers' methods could be found by the use of this formula seems very doubtful, inasmuch as Mr. Bauman omits the terms for moment and shear from the numerator while retaining the terms for moment and shear in the denominator. Since the arch is constructed of an elastic material, it must shorten under load, thereby setting up bending moments in the arch ring regardless of its shape. The complete expression for the elastic thrust at the crown of an elastic arch with all terms included is[26]:

$$H_e = \frac{-\sum_0^{\phi_1} M' y \,\Delta g - \sum_0^{\phi_1} \frac{P'}{t} \cos\phi \,\Delta s + 3\sum_0^{\phi_1} \frac{S'}{t} \sin\phi \,\Delta s - E\,\epsilon\,\Delta t \sum_0^{\phi_1} \cos\phi \,\Delta s}{\sum_0^{\phi_1} y^2 \,\Delta g + \sum_0^{\phi_1} \frac{\cos^2\phi \,\Delta s}{t} + 3\sum_0^{\phi_1} \frac{\sin^2\phi \,\Delta s}{t}} \quad \text{. . (90)}$$

[26] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1301.

If it is desired to omit the work done by shearing forces, the third terms of both numerator and denominator can be omitted. If the work done by bending moments were to be neglected, the first terms of numerator and denominator would be omitted. However, since the arch must be subjected to some elastic shortening[27] these moment terms must be included in both numerator and denominator and not dropped from the numerator and retained in the denominator as was done in Equation (82). Consequently, the writers are constrained to regard the stresses obtained for the "pressure line arch" by Mr. Bauman as being fictitious.

In conclusion, the writers wish to express their appreciation of the efforts of those who entered into the discussion of this paper.

---

[27] "Die Staumauern," von Dr.-Ing. N. Kelen, p. 13.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

GEORGE WASHINGTON BRIDGE GENERAL CONCEPTION AND DEVELOPMENT OF DESIGN

Discussion

BY MESSRS. GUSTAV LINDENTHAL AND HAROLD M. LEWIS

GUSTAV LINDENTHAL,[14] HON M. AM. SOC. C. E. (by letter).[14a]—That part of Mr. Ammann's very valuable paper that deals with the historical account of the events and plans, preceding the conception and construction of the George Washington Bridge seems to require some elaboration and explanation as far as it relates to the writer and the North River Bridge Company. The author begins the chronology of the attempts to bridge the Hudson River with a reference to an Act of the New Jersey Legislature in 1868, for the New York and New Jersey Bridge Company, and he follows it with an account of that Company's plans before Congress, for a bridge with a pier in the river, and the subsequent reports by two Boards of Engineers, appointed by the Government in 1894, on the practicability of a single span over the river.

How this Company came to make any plans at all after an interval of twenty-six years is interesting, and should not be omitted in an accurate historical account of bridging the Hudson River at New York. For an explanation, it seems appropriate to give first the genesis of the North River Bridge Company mentioned by the author in second place. In the fall of 1885 the late Samuel Rea, Hon. M. Am. Soc. C. E., at that time Assistant to the Vice-President of the Pennsylvania Railroad Company, discussed with the writer, on behalf of his road, the practicability of a railroad bridge across the Hudson River. The writer was not the only engineer so consulted, as Mr. Rea was a very able engineer with a penetrating and cautious mind. He analyzed the situation to the writer as follows: There was keen competition among the railroad companies for Western traffic. The New York Central

NOTE.—The paper by O. H. Ammann, M. Am. Soc. C. E., was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: November, 1932, by E. E. Howard, M. Am. Soc. C. E.; and January, 1933, by C. T. Schwarz, M. Am. Soc. C. E,

[14] Pres. and Chf. Engr., North River Bridge Co.; Cons. Engr., Jersey City, N. J.

[14a] Received by the Secretary November 26, 1932.

Railroad Company advertised a direct entrance, with four tracks, to the heart of Manhattan, while the Pennsylvania Railroad Company and the other railroads terminating in New Jersey were handicapped and had to transfer their passengers across the Hudson River by ferries. A tunnel under the Hudson River had been started at Hoboken, N. J., but it was intended only for small cars and local traffic. A larger tunnel for locomotives and standard cars appeared objectionable because of the smoke, which was then a subject of daily complaint in the tunnels of the New York Central Railroad. (Electric locomotives had not yet been invented.)

The great railroad bridge over the Firth of Forth in Scotland was then under construction. The question was, could a similar bridge be built over the Hudson River? A design for a railroad suspension bridge had just been discarded for the Firth of Forth Crossing, in favor of a rigid cantilever design. Evidently, a suspension structure, such as the Brooklyn Bridge or the Railroad Bridge over the Niagara River would not do. What was wanted was a rigid bridge for heavy locomotives and fast trains and with four tracks. This was the problem discussed in 1885. It was soon ascertained that the Government would not permit a pier in the river. In any case, the depth of rock (250 to 300 ft.) was prohibitive for piers. There must be a single span over the entire river, about 3 000 ft long. Could it be built?

The writer had given thought to the matter before, but he made further studies of the problem, and in the spring of 1886 presented a plan, and reported to the Pennsylvania Railroad Company that a rigid suspension bridge of four tracks and with a single span of 3 000 ft was practicable on a location near Desbrosses Street, New York City. The system for the trusses was that of suspended braced arches, known as the "garland" type, in which the chords were to be wire cables. The New Jersey approach was to be located over the Horsemus Cove Branch (owned by the Pennsylvania Railroad Company) opposite Desbrosses Street. The New York approach would turn north and descend to a terminal near Washington Square. The total cost for bridge and terminal would be about $22 000 000. This was a greater undertaking than the Pennsylvania Railroad Company at that time cared to assume. It was then considered that the bridge should be used by several railroads, that it should have six tracks, and should be located at 23d Street, with a terminal on Sixth Avenue, which was then the business center in Manhattan.

The North River Bridge Company was organized for that purpose in 1887, the writer was appointed Chief Engineer, and, in 1888, application was made to Congress for a franchise. The plans were explained to Gen. Lincoln N. Casey, then Chief of Engineers, U. S. Army, who recognized them as practicable. The charter was granted in July, 1890. At the behest of the local members in Congress who desired rapid transit over the bridge, the number of tracks prescribed in the Act was increased from six to not less than ten.

The suspension design for the 23d Street Bridge location (published in 1888) was the first plan for a bridge over the Hudson River at New York, and it was for a rigid railroad bridge at that. All other designs were proposed by later engineers, a fact which appears to be obscured in the author's introduction. The feature of a very long suspended span was at no time

the problem—this was always simple enough since it requires merely heavier cables—but the construction of a suspension bridge of long span, sufficiently rigid and economical for heavy railroad traffic at high speeds on several tracks was the actual problem, for the solution of which the writer proposed his design of the garland type. He is still convinced that it is the most economical of all types for that purpose. The feasibility of a near rigid suspension bridge for railroad loads and for a span of 3 200 ft without stiffening frame of any kind, but also its prohibitive cost, was discussed by the writer in his paper on "A Rational Form of Stiffened Suspension Bridge." [15]

The location of a bridge across the Hudson River at 23d Street was approved by the War Department in December, 1891, and work was commenced in 1892; but no sooner did the plans of the North River Bridge Company become known, than a rival bridge company sprang up with an old New Jersey charter, granted in the speculative era after the Civil War. Its scope and jurisdiction were supplemented with legislation in Albany, N. Y., in 1890. This Company was The New York and New Jersey Bridge Company described by the author in his introduction. It was advised by its engineers that the plans of the North River Bridge Company were impracticable for railroad purposes.

The argument quoted by Mr. Ammann against "spanning the North River without a pier,"[2] voiced the opinion then prevailing among leading bridge engineers, and was directed against the writer's plans. A rigid cantilever bridge with piers in the river was proposed, but the plan was opposed by the War Department. It led to the appointment of two separate Boards of Engineers by the Government, to investigate the practicability of a single span for six railroad tracks. By that time the construction of such a bridge had been authorized by the Government, and had actually been started on plans by the writer; but it was interrupted by litigation, to which the New York and New Jersey Bridge Company was a party, disputing the question of the right of condemnation of land claimed by the North River Bridge Company. It was decided by the U. S. Supreme Court in 1894 in favor of the North River Bridge Company. The reports of the two Boards of Engineers in 1894 are quoted by Mr. Ammann. Both reports discuss the plans of the writer, and one of these reports contains a detailed description of the North River Bridge of that time. All this happened in 1894, several years after the writer's plans were first publicly known.

The plans of a railroad bridge over the Hudson River, unlike those of a bridge for only highway or rapid transit, are inseparable from plans for railroad terminals and connections; therefore, the plans for the 23d Street Bridge included a terminal station on Sixth Avenue, and a rail connection with the Long Island Railroad, through the so-called Steinway Tunnel under 42d Street, which was then being built. Had the railroads combined with the Pennsylvania Railroad Company in 1890 to build the 23d Street Bridge, it would have caused an estimated addition of 1 500 000 population to Northern New Jersey by this time.

[15] *Transactions*, Am. Soc. C. E., Vol. LV (1905), p. 64.

[2] See Senate Ex. Doc. No. 12, 53d Cong., 3d Sess.

Subsequent events and a business depression, as related by the author, caused delays and led to changes in the plans. The exigencies of the World War intensified and congested traffic on the river and in the harbor. An exceptionally cold winter froze the ferry slips, and for an entire week interrupted the transfer of car-floats carrying food and fuel to Manhattan. The public demand for a bridge became more urgent than ever. Thereupon the North River Bridge Company, in consultation with the Chief Engineers of all the railroad companies (nine in New Jersey and three in New York) prepared the plans for a large freight terminal, which would be directly accessible to ocean shipping and from thirty intersecting streets in the warehouse district. The plans included also a large passenger station on Eighth Avenue.

The bridge was only about one-third of the entire project, which was judged to be the best conceived for its purpose. It had been developed in detail with the aid of railroad transportation experts. The combined traffic would require twelve tracks over the bridge (four for freight, four for passenger trains, and four for rapid transit), all on one level, which explains the cross-section (Fig. 4) given in the author's paper of the double-deck floor-beam, arranged on the Viereendel system. Highway traffic would find room as needed on the upper deck.

More recently, revised plans for the 57th Street Bridge and terminals, to meet the changed conditions, have been prepared. It is proposed to build the bridge in several stages, adjusted to the exigencies of transportation as they may develop for a roadway and heavy railroad structure, intended to endure many centuries.

HAROLD M. LEWIS,[15] M. AM. SOC. C. E. (by letter).[15a]—The George Washington Bridge over the Hudson River, as described in Mr. Ammann's paper, appeals to many because of its tremendous size and the physical difficulties overcome in its construction. It is also of great importance in its probable effect on the pattern of the future highway system of the New York Region.

Place in Regional Plan of Communications.—Earlier projects for a Hudson River Bridge at New York City were put forward primarily to provide a new connection between New Jersey and the metropolitan business centers on the Island of Manhattan. Public agitation for new transportation facilities is generally directed toward those that parallel already congested routes in the same vicinity. As a part of the regional highway system, the George Washington Bridge is essentially a way around the congested centers and, to this extent, it provides an entirely new facility. The Port of New York Authority, with its independent method of financing its projects, has been able to locate this crossing where it is most needed to improve the distribution of vehicle traffic in the entire Metropolitan District rather than where it would serve merely as another artery to encourage the continuance of a poor distribution. It is the major link in the proposed metropolitan loop highway in the Regional Plan.

[15] Cons. Engr.; Engr., Regional Plan Assoc., Inc., New York, N. Y.

[15a] Received by the Secretary December 6, 1932.

Importance of Rail Facilities.—The George Washington Bridge eventually should also become part of a transportation "corridor" around the central business and residential areas in the Metropolitan District. Such a "corridor" will provide, in places, for all types of ground transportation. From a planning point of view, to let rail facilities from New Jersey terminate in Northern Manhattan and compel transfer to the limited local transit lines in that part of the city, would be as faulty as to terminate the highway approaches there.

From its early studies the Staff of the Regional Plan of New York and Its Environs has considered a bridge at the site of the present one as an important link in the railroad, as well as in the highway, system of the Region. The importance of considering rail facilities in the design of the bridge was pointed out by the writer on behalf of the Regional Plan at a meeting of the New York and New Jersey Hudson River Bridge Advisory Committee of the Port of New York Authority on March 12, 1926.

At that time the writer emphasized the fact that a bridge at 179th Street would serve not only as immediate relief to traffic conditions, but would furnish an incentive for, and make possible the development of, any decentralization that seems desirable. Studies by the Regional Plan of New York and Its Environs had indicated that, in order to serve adequately in this way, the bridge and its connections should be designed so that eventually it could carry every possible kind of transportation that may flow over it in the future, including pedestrians, trackless vehicles, and railroad services of all kinds.

These recommendations were based upon a plan for regional rail facilities, which had been worked out under the guidance of William J. Wilgus, M. Am. Soc. C. E., who acted as Consultant on studies of transportation and port development for the Regional Plan. His recommendations for rail links between the bridge and both the New Jersey and New York railroads are incorporated in the Graphic Regional Plan.

The Regional Plan gave consideration to three kinds of rail transportation: Trunk-line railroads, suburban rapid transit, and local rapid transit. The way in which the George Washington Bridge might serve these three systems of rail transportation is shown in Fig. 25. It is realized that the use of the bridge for rail purposes involves the co-operation not only of the railroad companies, but also of many public agencies. As explained in Mr. Ammann's paper, the Port Authority Staff felt that "the prospective volume of [rail] traffic fully warranted the comparatively small expenditure which was necessary to provide for four rapid transit tracks on the bridge," which can be accommodated on a future lower deck. Even though it may be many years before rails are actually added to the bridge, it is not too soon to develop plans for suitable connections with them. The proposal to carry local rapid transit tracks across the bridge to link Hudson County with Manhattan may seem to some to be beyond the realms of possibility, due to political difficulties resulting from the State line in the center of the river. Perhaps the time may come when practical advantages may be given greater consideration than political expediency.

Estimates of Future Traffic.—The forecasting of future traffic on a structure which provides a new type of facility is a most difficult problem. Past trends are insufficient, as new traffic may be created by the new facility.

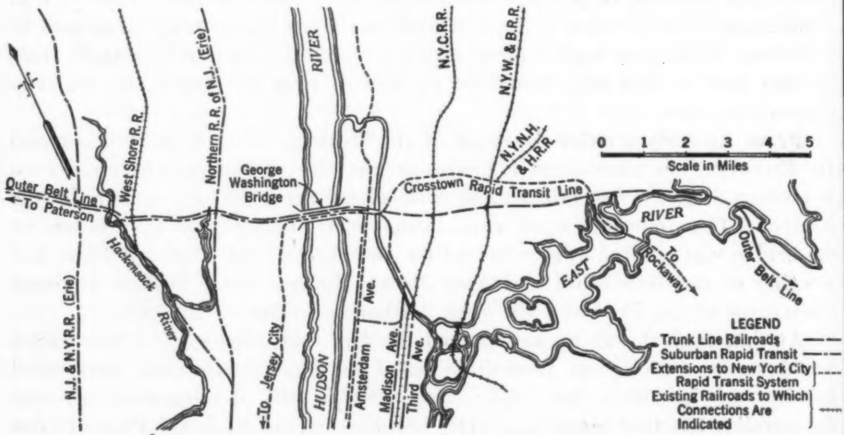


FIG. 25.—MAP SHOWING POSSIBLE RAIL USES OF THE GEORGE WASHINGTON BRIDGE OVER THE HUDSON RIVER.

The Regional Plan attempted to create a graphic picture of future vehicular traffic in the New York Region by the "distribution method." This was based upon sub-dividing the Region into a certain number of districts, each of which would have the same future population, and estimating the future traffic between each of these districts and every other district. The amount of traffic was considered a function of registration and distance. Based on a population of 21 000 000 in the Region in 1965, the vehicular traffic in that year between Manhattan and New Jersey was estimated at about 55 000 000. From this and past records up to and including the year 1926, estimates were prepared in 1927 for future trends in both the total traffic between Manhattan and New Jersey, and those parts of it which would be north and south, respectively, of 40th Street in Manhattan. These estimates up to 1950 are shown in Fig. 26 where they are compared with what has actually happened since 1926. The diagram also includes estimates prepared in 1927 for the minimum traffic through the Holland Tunnel and across the George Washington Bridge.

It appears in the diagram that, in 1931, traffic in the Holland Tunnel of 12 756 193 vehicles was about twice the minimum estimated by the Regional Plan. The new traffic created by this facility had shoved the curves of actual traffic well above the estimated curves both for the total between Manhattan and New Jersey and for those crossing the Hudson River south of 40th Street, Manhattan. During its fifth year of operation ending November 13, 1932, the Holland Tunnel carried 11 636 446 vehicles. This has been added to Fig. 26 and shows an 8.8% loss over 1931. The downward trend is undoubtedly only temporary, but has certainly held down the

traffic over the George Washington Bridge below that which would have occurred normally. Nevertheless, this traffic during the first year of operation of the bridge, ending October 15, 1932, and amounting to 5 628 234 vehicles, is fairly close to the Regional Plan estimate of 6 100 000 vehicles in 1932.

The writer believes that the 1950 estimates for total traffic between Manhattan and New Jersey and that over the George Washington Bridge, as shown in Fig. 26, still present conservative figures for that date. They

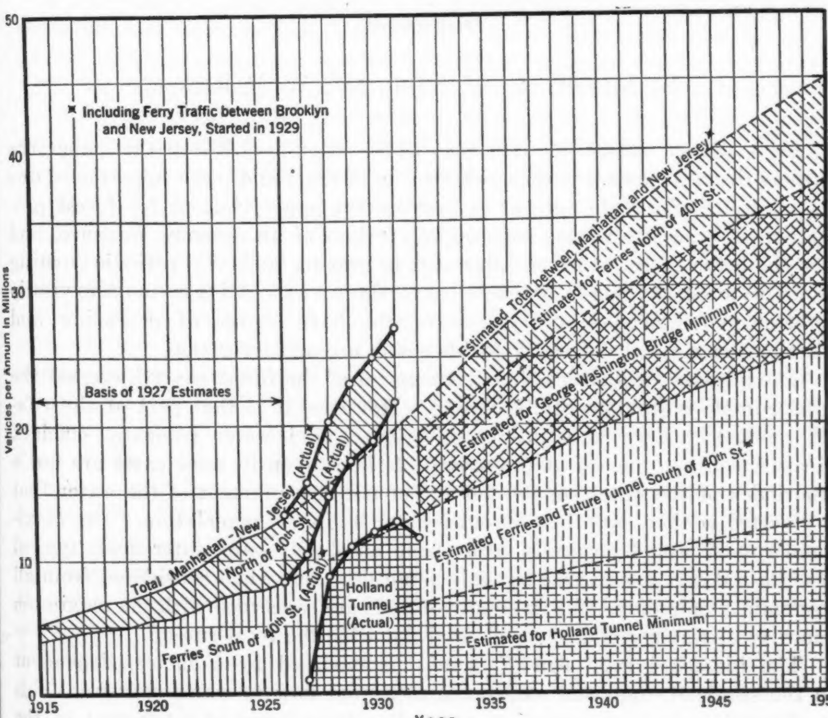


FIG. 26.—VEHICLE TRAFFIC ACROSS THE HUDSON RIVER AT NEW YORK CITY, 1915–1950. ESTIMATES MADE IN 1927 COMPARED WITH ACTUAL TRAFFIC, 1927–1931.

might be considered as minimum estimates which are likely to be exceeded, as is indicated by the estimates in Mr. Ammann's paper. The latter also include the Yonkers, Piermont, and Tarrytown ferries, which are omitted in the estimates in the accompanying diagram.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FORESTS AND STREAM FLOW

Discussion

BY MESSRS. C. W. SOPP, AND W. P. ROWE

C. W. SOPP,[48] ASSOC. M. AM. SOC. C. E. (by letter).[48a]—In this paper the authors discuss a question that is becoming of more and more importance, due to the desire of certain persons to increase the appropriations for forest protection, of others to reduce such appropriations as an economy measure, and also the belief of certain men interested in grazing lands that periodic burning of the water-shed cover is beneficial to forage growth. All these matters vitally interest public officials and engineers who have charge of procuring and distributing a supply of water for domestic use and irrigation.

The authors' deductions that the removal of the forest cover increases the total run-off is fundamentally correct. However, it is that part of the run-off rendered usable that interests engineers and water company officials. Measurements of run-off at the mouth of the canyon in some cases are not a true index of the water supply, but rather the measurement of the water that is diverted or that replenishes ground-water basins by percolation. The maximum run-off would naturally occur from absolutely bare water-sheds typical of certain desert mountains, but this is not desirable when considered from all economic standpoints. Water supply officials are concerned with the maximum quantity of water rendered usable and not with the total run-off.

The increased flood run-off should be modified by a factor, dependent on the silt load, that will reduce the measured flow to water content. This is of considerable importance as even after three years of settlement of the first year's samples, the authors found that the silt was about 45% of the total volume. During the flood the bulking would be greater.

There is a direct loss of flood run-off after denudation by fire or by deforestation through the impossibility of diverting the silt-laden waters directly on the lands. On a denuded water-shed the flashy run-off occurs during or immediately after precipitation and at a time when the lands are saturated from the rainfall and there is no need for irrigation. The side-hill erosion

NOTE.—The paper by W. G. Hoyt, M. Am. Soc. C. E., and H. C. Troxwell, Assoc. M. Am. Soc. C. E., was presented at the Annual Convention, Yellowstone National Park, Wyoming, July 6, 1932, and was published in August, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1932, by C. G. Bates, Esq.; November, 1932, by Messrs. J. E. Willoughby, and A. L. Sonderegger; and December, 1932, by Messrs. J. C. Stevens, Harry F. Blaney, Daniel W. Mead, Ralph R. Randell, H. K. Barrows, Donald M. Baker, Ralph A. Smead, and George H. Cecil.

[48] Asst. Engr., Pasadena Water Dept., Pasadena, Calif.

[48a] Received by the Secretary November 23, 1932.

deposits a large quantity of débris in the stream beds, which is sufficient in most cases to transform a rocky stream bed into a broad sandy one. In the course of a number of years all this deposition is moved out by the stream, causing great difficulties at head-works. The material is moved continuously in the form of material in traction, which rapidly fills all diversion works. This necessitates a waste of water in diversion either because it is possible only to skim the surface of the water or because of the necessity of flushing the settling basins. The sluicing and skimming losses may more than offset the gain in run-off. Great difficulty was experienced in attempting to divert even the clear water of San Dimas Creek, in Southern California, for a year or two after its partial denudation in 1919. In another case of diversion of water for domestic supply the material carried in traction into the head-works was 1 cu yd per hour per sec-ft of water. The capacity of the settling basin was such that the water had to be turned out frequently and the basin sluiced. For these reasons, in the absence of storage, there is actually a direct loss of water for irrigation and domestic use.

Reservoir storage is an absolute necessity to make usable the increased run-off from a denuded drainage area, but erosion causes a rapid loss of storage space. One-half the drainage area above one reservoir in Southern California was burned over, and the run-off of the following winter season, even though it was a year of sub-normal rainfall, caused a loss of 11% in storage space. The deposition in the reservoir was 65% of the total run-off during the first year. This appears an abnormally high percentage, but it is not out of line with the percentage obtained by the authors during the first year, as shown by their table of silt samples (Table 12).

On another Southern California stream a flood-control reservoir was constructed the fourth year after a water-shed was denuded by fire. The subsequent deposition behind the dam has caused a considerable loss of storage and has interfered with the operation of the outlet gates. This illustrates that even four years in this case was not enough for the recovery of the water-shed cover to prevent considerable erosion. In Southern California, as a rule, five years are required for complete recovery under the most favorable situation as regards soil and moisture supply. After each fire the fertility of the soil is reduced by the burning of the humus and by erosion. The succeeding vegetation is of a lower type, and this process, if continued, will leave water-sheds practically bare.

There is an admitted increase in the summer flow from denuded water-sheds. In semi-arid localities the moisture in the water-shed exists as capillary water in the soil on the slopes and as free or gravity water in the alluvium in the canyon and valley bottom-lands. Under these conditions the removal of the forest cover from the side slopes would not cause an increase in the summer run-off inasmuch as the total supply available is only capillary water, and the removal of the transpiration loss would not increase the stream flow. The removal of the forest cover from the bottom-lands, especially from those water-loving plants and trees that have their roots in free water, does reduce the transpiration loss and causes an increase in stream flow. It has long been the observation of foresters that the summer or low-

water flow increases on water-sheds after fires, but springs on the slopes do not so increase. This beneficial increase in water supply could be obtained by deforesting the bottom-land growth without any increase in erosion from the side slopes and probably very little increase in erosion of bottom-lands. The increased non-flood run-off is dependent on storage in the soil cover, but such storage would only be temporary because the continued practice of deforestation would eventually cause the removal of the soil storage.

In addition to the loss of water under conditions of direct diversion from denuded water-sheds and of the loss of valuable storage space in reservoirs, there is an indirect loss where an influent silt-laden stream traverses ground-water basins. The silt content tends to seal the stream beds and to reduce the percolation and recharge of these basins.

The ultimate result of continued denudation by fire or deforestation is to cause all run-off to occur in flashy floods when there is little demand for direct diversion for irrigation. It will also result in the erosion of soil cover that forms the reservoir from which the increased summer flow occurs, thereby removing the one beneficial influence derived from such denudation. Furthermore, it necessitates large and cheap reservoir space to make the increased run-off usable, and such storage is rapidly lost. The water-sheds are destroyed for all uses other than maximum run-off. Such denudation and deforestation should not be permitted, except where maximum total run-off is the highest use for which a water-shed has been classified, and cheap reservoir space is available.

W. P. Rowe,[49] Assoc. M. Soc. C. E. (by letter).[49a]—The major portion of the mountain drainage area, tributary to the South Coastal Basin of Southern California, is covered with chaparral or brush. The true timber area comprises the lesser portion at higher levels. This area is used almost exclusively for recreational purposes and is visited each year by more people than any National Park in America. These two classes of vegetation are rarely found intermingled, probably because the rains are not sufficient to support both. Either will predominate to the exclusion of the other. No person has ever advocated the destruction of the true timber area in order to increase or control the water supply; therefore, this class will be omitted from the discussion to follow, the writer's remarks being confined to the chaparral areas of Southern California.

The Forest Service of the U. S. Department of Agriculture classifies chaparral as "forest cover," and this classification leads to confusion among engineers in the true timber localities, who are not familiar with the distinction between the two. Until the last few years, the Forest Service referred even to the most insignificant brush fire as a "forest fire." The local newspapers were of great assistance in spreading this misleading information. There is as much difference between burning the chaparral and timber as between paring a fingernail and cutting off a finger, except that burning of the timber is soon offset by a rapid growth of chaparral, whereas a finger can never be replaced.

---

[49] Cons. Engr., San Bernardino, Calif.

[49a] Received by the Secretary November 30, 1932.

Chaparral growth follows a regular and never-ending cycle: First, comes the natural growth; then, a fire, started by any number of causes, burns off the chaparral. Then, follows the dire prediction of the professional conservationists that the water-shed is destroyed and the water supply ruined. The chaparral starts to sprout as soon as the fire is quenched. Then occur the winter rains and a little more soil is added to the valley floor and considerable more water is added to the underground basins that regulate 95% of the local water supply. The chaparral growth is back to normal in a few years, and the cycle is complete. These cycles have been occurring since the first lightning flash struck the dry chaparral, and they have occurred at more frequent intervals since the first Indian discovered that a chaparral fire resulted in more abundant feed for game and that it made hunting easier.

Every stream water-shed consists of a series of still smaller water-sheds each of which is traversed by a drainage channel. These drainage or stream channels accumulate the surplus water reaching the water-shed and convey it to the main streams. They are usually bordered by water-loving plants and act as natural fire-breaks. The steep side hills are quickly drained of surplus water and the end of each summer generally finds a veritable tinder-box awaiting the least spark to start a typical chaparral fire. There is a school of thought in California, composed principally of old-time settlers, *ex*-forest rangers, and hydraulic engineers versed in local water supplies, which advocates periodic and systematic burning of small areas of the chaparral. By burning the canyon ridges at alternate periods of ten years, there would be not more than 50% denudation in any small water-shed and there would be less than 10% denudation of a major chaparral-covered water-shed at any one time. The period of ten years is used only for the sake of comparison. It might be found that either a shorter or a longer interval was better.

All the arguments that have been advanced against such a plan of controlled burning can be applied tenfold against the results of the present policy of protection. The fire that occurred in Ventura and Santa Barbara Counties in 1932 burned over entire water-sheds, and it is estimated that more than \$100 000 was spent in trying to control it. A shift in the wind and accompanying fog extinguished this fire. It is estimated that more than \$250 000 was spent in trying to control the San Gabriel fire in 1924, but a fog quenched this fire also.

The proposal of controlled burning of chaparral areas is not advanced as a measure for increasing the water supply, but as a simple method of fire protection to the recreational area and lessening, by at least 90%, the evils arising under the present system. The areas of most recent burning would be refuges for the game that is killed under the present system of maintaining areas of brush too thick and too extensive for quick evacuation in case of fire. The authors have exploded the theory that a burned-over area is destroyed as a water producer. It must not be overlooked, furthermore, that the very conditions that made it possible for the authors to draw their conclusions for chaparral-covered areas, were the result of a fire that did occur under the present policy.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TESTS OF RIVETED AND WELDED STEEL COLUMNS

Discussion

BY MESSRS. R. A. CAUGHEY, E. G. WALKER, AND M. HOLT AND R. G. STURM

R. A. CAUGHEY,[11] M. AM. SOC. C. E. (by letter).[11a]—It occurs to the writer in reading this paper that it brings out the fact that there is much to be learned regarding the effect of welding in steel. There seems to be abundant evidence that permanent deformation is one of the results of welding. In Conclusion (2) the authors note that stitch-welding caused shortening of the metal at sections through the welds and elongation at the edges of the cover-plates at sections midway between welds. They also show that the moduli of elasticity of the columns as tested were lower than those obtained in coupon tests. It would seem that there may be a definite relation between these two phenomena; namely, that on account of unequal shrinkage in cooling, certain parts of the column sections received an initial compressive stress and other parts some initial tension. It is possible that when load was applied, the compression sections began to take compressive stress immediately, but that the sections in tension did not, until their initial tension had been overcome. Naturally, this would cause more deformation in the column than would exist if the entire section was acting; and, therefore, would give a lower modulus of elasticity for the compound section.

As is stated by the authors, the two moduli do not differ radically, but the difference seems to indicate that initial stresses due to welding do exist and that they affect the modulus in the manner mentioned. Referring to Table 5, it will be noted that the moduli for the two riveted columns, Nos. 2 and 5, are 28 800 000 and 29 100 000, respectively, while those of welded columns (omitting Column No. 3 which seems to discount to some extent the writer's hypothesis) vary from 26 000 000 to 28 400 000. It should also be noted in connection with this table that the determination of the moduli, in cases of Columns Nos. 1, 6, and 7, is not entirely satisfactory to the authors.

NOTE.—The paper by the late Willis A. Slater and M. O. Fuller, Members, Am. Soc. C. E., was published in September, 1932. Discussion on the paper has appeared in *Proceedings* as follows: January, 1933, by Messrs. Raymond J. Roark, and Lyndon F. Kirkley.

[11] Prof. of Structural Eng., Iowa State Coll., Ames., Iowa.

[11a] Received by the Secretary December 5, 1932.

Recently the writer had a little experience with the effect of welding on some small **H**-columns. It was desired to weld some small bars to the columns for the purpose of carrying clinometers to measure rotation of joints. In this case oxy-acetylene welding was utilized, and it is likely that a larger amount of metal was heated than would have been the case in electric welding. It did not take long, however, to find out that all strain-gauge holes near the welding had to be re-drilled, as the gauges would no longer fit into them. The welding process was abandoned in favor of tap-bolts.

If it was possible to do so, it would seem advisable to test some short lengths of compound sections, the specimens to be arranged in such a way that welds could be released without re-heating and stresses measured in various sections separately. It might be possible in this way to get some ideas as to initial stresses in the various shapes of the section. It might also be advisable to make some coupon tests from material adjacent to, or forming, welded connections in order to obtain data on possible change of moduli.

It is possible that changes in moduli may be caused by uneven flow of metal in various parts of a compound section after welding is completed. For instance, perhaps the metal at the junction of the web and flange of the **H** or **I**-sections of the columns under consideration would be somewhat more resistant to deformation than that in the outer ends of the plates riveted or welded to them. Fig. 16(*a*) seems to indicate that this is true in the tests under consideration.

The writer feels that more should be known regarding moduli changes and initial stresses caused by welding. Testing such sections as were tested by the authors, with provision for releasing welds and making coupon tests of material that has been welded, would probably give some information of that kind.

E. G. Walker,[12] M. Am. Soc. C. E. (by letter).[12a]—The writer has read this paper with considerable interest and hopes that the comparison between riveted and welded structural steel work instituted therein will be continued. The paper evidences the considerable care and attention to detail that has been bestowed upon the work. It shows also that in the study of so uncertain an article as a composite structural steel column it is difficult to draw conclusions except after testing a large number of specimens. The investigation in the present case has been brought to a stage that enables one to judge the main structural differences resulting from the adoption of welding in place of riveting for built-up columns. It is insufficient, however, to enable definite distinctions between the two types to be formulated.

In the tests on the slipping of plates, no instance of undoubted slip of definite magnitude occurs. The fact that all the types tested behaved nearly alike does not enable logical conclusions to be drawn. Unfortunately, no tests are available in which slip actually occurred. Had there been, it might have been possible to deduce conclusions therefrom as to the relative resistance of the riveted and welded columns to slipping. Further experiments are

[12] (Maxted & Knott), London, England.

[12a] Received by the Secretary December 12, 1932.

necessary before it can be decided whether, if slip should actually occur in a riveted column, it would also take place in a corresponding welded column.

The use of welding, by reason of the localized heating of the structural material inherent in the process, introduces a new factor which does not have to be considered in connection with riveted work, where, although there must be a limited amount of localized heating around the rivet hole, this heating cannot extend to a sufficient volume of metal to influence stress distribution. The authors' investigations indicate that a certain amount of deflection is caused by stitch-welding and, therefore, there must be a corresponding amount of initial stress in the material. The question arises, as to whether, if heavier and more complicated sections than those used in the experiments were built up by welding, there might not be the possibility of developing initial stresses of such magnitude as to be a source of structural weakness. This does not seem likely, but it is a point that might well be investigated in the future.

The results of the deflection measurements are somewhat remarkable, in that they indicate a greater deflection in the case of the continuous welded column than in the other two cases. The writer would have expected the relative magnitudes to be the other way round, if there was any difference at all. It is quite possible that the limited number of tests available in this case have induced erroneous conclusions. The authors' strain measurements have led them to conclude that there was no definite slip in any of the columns. If that is so, there would not appear to be any reason for variation in the amount of deflection, and it is possible, therefore, that the conclusion may have been arrived at because the number of specimens of which records are available, is insufficient to enable true relative averages to be obtained.

The comparison of actual and calculated maximum loads given in Table 3, is of subsidiary interest. The values of the ratio of these stresses are thoroughly consistent, and the range of variation is not greater than one would expect. It follows, therefore, that the Rankine formula, with the constant used by the authors, gives definitely a strength lower than the actual strength of the specimen. An attempt made to adjust the constant in the denominator of Equation (1) which is the only constant that can be varied (the factor, S, being determined directly from the experimental results), was not altogether successful, and showed that, instead of being $\frac{1}{25\,000}$ as given, the figure should be about $\frac{1}{65\,000}$. This is inconsistent with the generally published values of this constant. It would appear that there is some other factor that should be considered in relation to the test results. Probably the degree of fixity of the ends of the columns when set up for test is the controlling factor.

M. HOLT,[13] JUN. AM. SOC. C. E., AND R. G. STURM,[14] ASSOC. M. AM. SOC. C. E. (by letter).[14a]—The deflections of columns with known eccentricities

[13] Research Engr., Aluminum Research Laboratories, New Kensington, Pa.

[14] Research Engr.-Physicist, Aluminum Research Laboratories, New Kensington, Pa.

[14a] Received by the Secretary December 23, 1932.

have been measured by the writers and have been found to agree closely with those computed by Fidler's formula:[15]

$$D = \frac{Pe}{P' - P} \quad \text{.........................} (2)$$

in which, D is the deflection of the column, in inches; P, the load on the column, in pounds; P', the critical load according to Euler, in pounds; and e, the total initial eccentricity, in inches.

A study of Equation (2) shows that the load-deflection curve for a column is curved from the first, thereby making the selection of a proportional limit from the load-deflection curve very indefinite. This probably contributed to the difficulty in selecting the proportional limits for the columns tested by the authors.

All columns have some eccentricity, which may be accidental and unintentional and may be made up of the sum of the effects of crookedness, non-homogeneity of material, uneven bearing at the ends, etc. The initial eccentricity will be large or small depending on whether the foregoing effects are all in the same direction or whether they counteract one another. Some of the columns reported in this paper were apparently quite straight while Specimen No. 4, for example, appears to have been quite crooked. The total initial eccentricity can be found from load-deflection data by plotting $\frac{1}{D}$ against $\frac{1}{P}$ and interpreting the data in the light of Fidler's formula which may be re-written as:

$$\frac{1}{D} = \frac{1}{P} \times \frac{P'}{e} - \frac{1}{e} \quad \text{.....................} (3)$$

Equation (3) was first used in this form by Professors W. E. Ayrton and John Perry.[16] It may be noted that the intercept of this straight line on the $\frac{1}{D}$-axis is $\frac{-1}{e}$, and the intercept on the $\frac{1}{P}$-axis is $\frac{1}{P'}$. Information picked from the plotted data given in Fig. 18 of the paper is not sufficiently accurate to yield a reliable determination of the initial eccentricities of the specimens.

The fact that the bending stresses reported in the paper seem to have little effect on the maximum loads carried by mild steel columns is referred to as strange. It may be noted that in the cases shown, the bending stresses were relatively small and that if they are added to the ultimate strengths of the columns the total stresses would be quite close to the yield point stresses. Because of the definite yield point of the steel in the columns, the maximum stresses would not have an opportunity to increase to values appreciably above the yield strength of the material without excessive bending deformations. Thus, the loads giving maximum computed stresses equal to the yield strengths agree closely with the values obtained from the tests.

[15] Fidler's "Treatise on Bridge Construction."

[16] First used by Prof. W. E. Ayrton, and Prof. John Perry, in their paper, "On Struts," *The Engineer* (London), December 10 and 24, 1886.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

THE MARTINEZ-BENICIA BRIDGE

Discussion

BY JACOB FELD, ASSOC. M. AM. SOC. C. E.

JACOB FELD,[6] ASSOC. M. AM. SOC. C. E. (by letter).[6a]—The method of sinking caissons for the deep-water piers is one of the most noteworthy items in the design and construction of this bridge. Although it is not a new idea, the development of the technique and the success of the operation should result in the standardization of open-caisson construction where depths of water are not excessive.

The use of an artificial sand island for each pier eliminates the dangers from tide and current in making difficult the sinking of a caisson and also provides an easy method for the construction of the cutting edge in true location.

The two main difficulties in caisson construction have been floating the pre-cast or lowest section (usually built on shore) and the danger from tipping during sinking. The use of a sand island eliminates both dangers. In addition, careful explorations can be made of rock contours beneath the proposed caisson and the cutting edge can be built to suit such conditions, which practically guarantees a perfect fit when the caisson reaches rock.

Of course, when depths of water are excessive the cost of an artificial sand island may be prohibitive. In addition, the skin friction induced by the sand that has been placed artificially, increases the cost of sinking the caisson. This method might be revised in deep-water conditions by providing a steel shell enclosure driven to mud, and using such shell as the foundation for a working platform upon which the cutting edge can be set, the lower part can be built, and the entire caisson floated therein. As a matter of record such a procedure has been used for the Grey Strait Bridge, at Brisbane, Queensland, Australia, described subsequently.

As far as the geology of the ground is concerned a comparison of the sub-surface investigations reported by Mr. Kirkbride, with the result of similar investigations made for the Carquinez Strait Bridge, shows almost identical conditions.[7]

NOTE.—The paper by W. H. Kirkbride, M. Am. Soc. C. E., was presented at the meeting of the Structural Division, Sacramento, Calif., April 24, 1930, and published in September, 1932, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1932, by Messrs. N. F. Helmers, E. J. Schneider, and M. F. Clements.

[6] Cons. Engr., New York, N. Y.

[6a] Received by the Secretary November 22, 1932.

[7] "Design of Substructure of Carquinez Strait Bridge, California," by Jacob Feld, Assoc. M. Am. Soc. C. E., *Proceedings*, Brooklyn Engrs. Club, Vol. 23, April, 1925, pp. 3-34.

The nature of the rock found at that location was the same as at the Martinez-Benicia Bridge. In addition, it was found that at the Carquinez Strait Bridge layers of sandstone and shale were often separated by beds of softer material, such as clay and in some cases gravel. It was also found that the deeper water was not above the old filled-in channel. As a matter of fact, Pier No. 4 of the Carquinez Bridge, which is above the original deep channel, is founded on piles because sound rock was not found at a depth of 200 ft. below mean high water.

Within the ten years, 1922-1932, there have been some noteworthy examples of deep caisson construction, including at least ten examples of foundation construction in which caissons were sunk more than 100 ft. below either the ground level or the water level. A short description of each of these should be of interest because they show the rapid development of this type of construction.

(1) In 1923, for the Rigolets Bridge, of the Louisville and Nashville Railroad Company, about 30 miles east of New Orleans, La., five steel shell caissons, 34 ft. in external diameter, with single dredging wells 16 ft. in diameter, were sunk to total depths of 80 to 113 ft. Under 45 ft. of water there was a considerable number of layers of soft black mud overlying stiff yellow clay, fine sand, and some gravel. These caissons were floated into a ring of steel sheet-piling partly driven before the caissons arrived, and the remainder of the ring was completed after the caisson had been located. The caissons were sunk entirely by open dredging methods and by the addition of concrete between the two steel rings that form them.

(2) In 1923, at Lexington, Mo., two steel cylinders, 14 ft. in diameter, were sunk to a total depth of 110 ft. Under 13 ft. of water there were various layers of sand, blue clay, gravel, and boulders. The caissons rested on soapstone. Most of the excavation was done under air, a maximum pressure of 50 lb. being used.

(3) In 1926, for the Carquinez Strait Bridge, in San Francisco Bay, caissons with a dimension of 40 by 40 ft., and 112 ft. in length, were floated into position and sunk by open dredging methods. They were built of timber with a steel covering for the cutting edges, but with concrete posts so that, in the event the timber surface was removed the timber cross-bracing would not be exposed to the action of salt water and wood borers.

Before floating the caissons into position a guide frame consisting of six steel spuds, 120 ft. long, was built up and anchored by four anchor barges. The conditions at the pier were 80 to 90 ft. of water overlying various layers of mud, coarse sand, gravel, and thin layers of clay, as well as shale and sandstone. The caissons were sunk to a maximum depth of 135 ft. below water level. It was noted that the friction-reducing jets placed on the exterior faces of the caissons were not of much value, whereas the cutting jets placed on the interior faces of the caissons were useful in assisting the sinking.

(4) In 1927, for the Kennebec Bridge, at Bath, Me., three of the seven caissons were sunk more than 100 ft. Two of these were 22 by 40 ft. and the third was 25 by 54 ft. The caissons were of wood, built 20 ft. high, floated, and held in position by six Chinese anchors. The excavated material was

chiefly plastic blue clay and coarse sand. The caissons rest on granite rock. The excavation was done chiefly under air.

(5) In 1927, for the Umgeni River Bridge, in South Africa, six caissons, 12 ft. in diameter, were sunk to depths of 60 to 156 ft. Because of the extreme low water, sections 20 ft. in height were built in place and sunk through coarse sand and clay by open grab-bucket excavation. The caissons were built with 2-ft. walls and steel cutting edges and sealed by tremie concrete.

(6) In 1928, for the Detroit River Suspension Bridge, between Detroit, Mich., and Windsor, Ont., Canada, two caissons, 38 ft. in diameter, were located in water varying from 8 to 25 ft. in depth. At these locations, artificial islands were built by pumping sand to above water level, and the caissons were started on top of this fill. In addition to these caissons, there were two other caissons 38 ft. in diameter and two anchorage caissons each 100 by 225 ft. These caissons were sunk by open dredging through mud and sand and then blue clay about 75 ft. in thickness, and varying layers of quicksand from 20 to 62 ft. in thickness. When the caissons reached hardpan (a layer approximately 10 ft. thick, overlying limestone rock) that material was excavated under air, and the caissons were sealed under air.

(7) In 1928, two caissons were sunk for the Mid-Hudson Bridge, at Poughkeepsie, N. Y. Each caisson was 60 by 136 ft., with a steel cutting edge and a steel shell 20 ft. deep. A temporary wooden bottom had to be installed for floating, and the caisson was held in position by eight 15-ton anchors. The conditions were 60 ft. of water and 18 to 35 ft. of mud overlying stiff clay, sand, and gravel, the east caisson going down to Elevation − 114 and the west one to Elevation − 134. All material was excavated in the open, and considerable difficulty was encountered as has been explained in recent engineering literature.[8]

(8) In 1929, for the Federal Barge Line Terminal, on the Mississippi River, several circular caissons, 10 and 12 ft. in diameter, with 2 ft. 0-in. and 2 ft. 3-in. walls, using steel cutting edges and steel shell forms, were sunk through 80 ft. of water and 40 ft. of sand with some blue clay. In these long and narrow caissons it was found that a balanced pressure must be maintained to prevent tipping. Sinking was performed by clam-shell excavation with the assistance of jets on the outside of the caissons.

(9) In 1929, for the Grey Strait Bridge, in Brisbane, Queensland, Australia, four caissons, 28 ft. in diameter, were sunk from 96 to 115 ft. through artificial sand-filled islands consisting of steel sheet-piling driven on a diameter of 30 ft. The first section of the caisson was built on this artificial sand island and was sunk by open dredging until rock was reached. The caisson was then sunk an additional 5 ft. into rock under air, using a maximum pressure of 50 lb.

(10) The latest example is that of the sixteen caissons sunk inside of 81-ft steel cylinders described in this paper, and it will be seen how the difficulties of the previous constructions have been eliminated by the methods described so ably by the author.

[8] *Engineering News-Record,* February 12, 1931, pp. 275–281.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TESTS FOR HYDRAULIC-FILL DAMS

Discussion

By Joel D. Justin, M. Am. Soc. C. E.

Joel D. Justin,[22] M. Am. Soc. C. E. (by letter).[22a]—The author has performed a valuable service to the profession in making available the laboratory control methods utilized on the Cobble Mountain Hydraulic-Fill Dam. It is to be feared, however, that many engineers not previously acquainted with these methods will gain the impression, on reading Mr. Hatch's paper, that the laboratory control there practiced was unduly elaborate and expensive, and, therefore, not applicable to work which they might be contemplating.

The writer knows that such an impression would be erroneous. The cost of the laboratory equipment utilized was not great, and the total cost of laboratory control was only a small part of the cost of engineering supervision and only an insignificant percentage of the cost of the structure. Accordingly, the writer believes that it will help to promote the use of laboratory control methods similar to those utilized at Cobble Mountain, if Mr. Hatch will give some data on the cost of the laboratory, the number of men employed in this branch of the work, and the total cost of laboratory control for the job, together with the cost per cubic yard of material in the dam. Such data would also help engineers responsible for the design and construction of earth dams in persuading their principals that it is worth while and not unduly expensive to have a thorough laboratory control on such work.

The magnitude, character of materials available, and type of dam used, should govern the degree of elaborateness adopted for laboratory control. With the advantage of the pioneer work in this line done by the late Allen Hazen, M. Am. Soc. C. E., and Mr. Hatch at Cobble Mountain Dam, and by a few others elsewhere, the necessary cost of laboratory control for most earth dams may be quite insignificant. Thus, at the Bee Tree Dam,[23] a

Note.—The paper by Harry H. Hatch, M. Am. Soc. C. E., was presented at the Joint Meeting of the Irrigation and Power Divisions, Yellowstone National Park, Wyoming, July 7, 1932, and published in October, 1932, *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: January, 1933, by Charles H. Paul, M. Am. Soc. C. E.

[22] Hydro-Elec. Engr., U. G. I. Co., Philadelphia, Pa.

[22a] Received by the Secretary December 8, 1932.

[23] "Earth Dam Projects," p. 94, John Wiley & Sons, Inc., N. Y., 1932.

hydraulic-fill dam, in North Carolina (177 ft. high), Charles E. Waddell, M. Am. Soc. C. E., utilized a sufficiently precise method of laboratory control for the existing conditions at a cost for laboratory equipment of less than $750, and one laboratory man and an assistant did practically all the work required.

In his Fig. 2, the author shows for a particular case, the percentage of voids in the core and in the beaches at various distances from the center line. The percentage of voids in the core is shown to exceed 50% slightly, the voids decreasing with the distance from the center line until at about 240 ft. the voids in the beaches are 25% on one side and 33% on the other side of the center line. In spite of this, the beach, because of the tremendously greater size of pores, was undoubtedly much more pervious than the core. The author does not state the depth below the surface of core pool or silt line at which the core samples were taken; nor does he state how long the core material, which had more than 50% of voids, had been in place.

One of the most compelling reasons for having a thorough laboratory control system in hydraulic-fill earth-dam construction is to obtain data that will enable one to control the stability of the core during the critical period while construction is in progress. The author gives a formula (Equation (24)) for determining the ultimate percentage of voids in the local core material at any depth, and another formula (Equation (30)) for determining the time it takes to secure this ultimate percentage of voids. He does not, however, give the percentage of voids in the core, which it is necessary to secure in order to have a stable core, nor the length of time, height, or pressure of material above, required to obtain this stable core. By a stable core is meant a condition of the core such that it is practically self-sustaining, and, therefore, does not exert material pressure against the retaining beaches or toes. The writer believes that the degree of stability of the core at different elevations during the progress of construction is one of the most important things that the engineer charged with the responsibility for the design and construction of a hydraulic-fill dam wants to know.

At the Calaveras Dam investigation, Mr. Hazen found that core material having 50% of voids exerted practically full hydrostatic pressure, but that core material having 40% of voids was sufficiently stable. The material in the Cobble Mountain core was much coarser than that at Calaveras, and it would be interesting to know at what percentage of voids the material became sufficiently stable.

Under "Seepage Formula for Hydraulic-Fill Dams," Mr. Hatch states that, so far as he knows, "there is no such general formula giving the seepage through any hydraulic-fill dam; * * *," and then proceeds to develop a formula for seepage through the core of such a dam. He is not entirely correct in this statement. A formula that may be used for this purpose was developed by Mr. Hazen, and is given by Mr. Hatch as Equation (22). The writer has previously given formulas for seepage through earth dams and also for seepage through the foundation soil.[24] These formulas are merely

[24] "Earth Dam Projects," pp. 139 and 174.

adaptations of the well-known Slichter formula to the conditions existing in earth dams. All these formulas, as well as that of the author, are based on the Darcy formula first presented in 1857 and given by Mr. Hatch as Equation (19).

The thoroughness with which laboratory control methods were developed and applied at Cobble Mountain have not been equalled elsewhere, as far as the writer is aware, and the detailed discussion of these methods given by Mr. Hatch will be extremely useful to all engineers engaged in the design and construction of earth dams.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

A HISTORY OF THE DEVELOPMENT OF WOODEN BRIDGES

Discussion

BY MESSRS. JASPER O. DRAFFIN, E. K. MORSE, AND JOHN W. STORRS

JASPER O. DRAFFIN,[42] M. AM. SOC. C. E. (by letter).[42a]—Timber bridges of the type discussed in this paper are fast disappearing, particularly in Vermont where a large number have been replaced within the last ten years and especially since the flood of 1927. The writer is particularly interested in one of the bridges described, the Enoch Hale Bridge, at Bellows Falls, Vt., or the first bridge over the Connecticut River. The latest successor to this primitive bridge is a two-span, reinforced concrete arch built in 1930.

That this early bridge was considered important is shown by the account mentioned by the authors,[43] which reports that Enoch Hale "has just built" a two-span bridge, across the Connecticut River at the "Great Falls," 360 ft. long and 60 ft. above high water. It was believed to be the most elegant, the strongest, and one of the most useful bridges in America. The item went on record that while the work of building it was dangerous, the only casualties were the death of a young man who fell on the rocks in the river, and a slight injury to Colonel Hale.

The main route between Boston, Mass., and Montreal, Que., Canada, passed over a turnpike road through Keene, N. H., crossed the Connecticut River at Bellows Falls, and continued across the Green Mountains to Rutland and Burlington, Vt., and thence to Montreal by Lake Champlain. Consequently, it was an important artery of travel and transportation at that time and the Hale Bridge was a vital link in it. The reason for the selection of this place for the construction of the bridge was that the river is narrow (300 to 400 ft. wide), passing through a rocky gorge with a ledge in the middle, on which a center pier could be placed. A more favorable site for a bridge can scarcely be imagined; but not only was the location desirable from an engineering point of view, it had a considerable reputation for scenic beauty.

NOTE.—The paper by Robert Fletcher and J. P. Snow, Members, Am. Soc. C. E., was published in November, 1932, *Proceedings*. Discussion of this paper has appeared in *Proceedings*, as follows: January, 1933, by Henry B. Seaman, M. Am. Soc. C. E.

[42] Assoc. Prof., Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

[42a] Received by the Secretary December 2, 1932.

[43] *The Massachusetts Spy*, February 10, 1785.

A question has been raised concerning the replacement of the original Hale Bridge. This arose mainly because Timothy Dwight stated[44] that, "in 1803 [the year of his first visit to Bellows Falls] the bridge erected by Colonel Hale * * * had been taken down and a new one built." He does not say that the replacement took place in 1803, but merely that it had been done by that time. In an earlier paragraph, he states that the original bridge was standing in 1797. Therefore, if his account is correct, a new bridge was built between 1797 and 1803 to replace the one built by Colonel Hale. Some doubt of the correctness of the Dwight account has been expressed, partly because there did not seem to be any local tradition of the replacement. However, there is a record that the owner of the bridge, a Mr. Guier (or Geyer), was trying to raise money by mortgaging his property at about the time of the supposed second building of the bridge. The bridges built by Colonel Hale have been described as a combination of arch and truss and neither the first bridge (see Fig. 8), nor the second one was of this type; but the description of the second bridge by Timothy Dwight would lead one to conclude that it was of the arch type, since he described it as "an obtuse arch." Taking into account all the uncertainties and the short time between the building of the first bridge and its replacement or rebuilding, it seems reasonable to conclude that the original bridge was not replaced in its entirety by a new bridge but that it was merely closed to traffic and extensively repaired between 1797 and 1803. In the beginning of his search for information, the writer wished to believe that there was an entire replacement of the bridge, but the evidence obtained thus far does not warrant such a conclusion.

The reconstructed bridge lasted until 1840 when it was replaced by what was known locally as the "Tucker Toll Bridge," shown in Fig. 18, and which was of the covered lattice timber type with sidewalks on the outside of each truss. It was placed 15 ft. higher than the Hale Bridge which was used during the construction of the new one, after which the old bridge was cut away and allowed to fall into the river.

E. K. Morse,[45] M. Am. Soc. C. E. (by letter).[45a]—It is refreshing to read something that is not buried in a mass of intricate calculations that are known only to the author. Several facts have not been mentioned in this paper, namely, that many of the old wooden bridges, especially those constructed by Burr, were built of sized white pine, free of knots, adzed into shape, and then hand-surfaced on four sides. This was especially true in the Burr Arch Bridge across the Allegheny River at Ninth Street, in Pittsburgh, Pa. (which the writer contracted to remove in 1890-91), and also in a bridge of the same design built by Burr at Saltsburg, Pa., across the Kiskiminetas River.

In inspecting the bridge at Saltsburg some years ago, the writer found the name of one of the carpenters who evidently had planed the beautifully shaped and finished members of the arches. The man had engraved his name

[44] "Travels in New England and New York," 1803.

[45] Engr. Member, Water and Power Resources Board of Pennsylvania, Pittsburgh, Pa.

[45a] Received by the Secretary December 10, 1932.

in the top of one of the arches which was just as bright and as clear as the year it was erected. All the timbers going into these bridges were seasoned from two to four years before being placed in position, which accounts for the accurate work and the condition of the bridges after years of heavy travel. On the Ninth Street Bridge there was not an open joint on any of the spans, averaging from 178 to 210 ft, center to center of skew-backs. The white oak flooring, like the white pine, was seasoned in the old-fashioned way and surfaced on all four sides. Teams and wagons passing over the floor moved so smoothly that the edges were not frayed. In practically all the bridges that were erected by Burr, the floors were originally laid dust-proof and because they were not exposed to the elements, they remained intact for many years.

In taking down the old Ninth Street Bridge, heavy vehicle and foot travel had to be maintained and, in order to facilitate erection, the roof was removed, the trusses were cut to the line of the arch, and $1\frac{1}{2}$-in. bolts were placed through the edge, and the verticals supporting the floor system clamped tight to the arches wherever they showed signs of weakness. In order to prevent excessive vibration, a two-post tower with sway-bracing was erected at the center of the arches. There was considerable vibration in the arches when they were cut loose from the floor system and the tower was removed, but when $1\frac{1}{2}$-in. manila rope was connected to the top of each arch and it was attempted to pull them over and dump them into the river, they yielded readily for about 3 or 4 ft and then resisted even the 5 tons of horizontal pull that was put on them, and eventually had to be removed panel by panel.

It was found that the members in all the arches had completely lost their fiber to the extent that it was almost impossible to cut one of these timber arches with an axe. It was as difficult to lift a chip from one of these white pine timbers as it is with a red gum or green sycamore. The timbers were bright. During the erection of the new bridge, they supported a load three or four times as great as that for which they were designed and, in no case, did any member of the arches show any sign of weakness before, during, or after being taken down. A match factory in Chicago, Ill., had bought the timber in the white pine arches, and needless to say the contract was cancelled when it was discovered that white pine which is so easy to chop and so beautiful to work, had become as tough as gum.

JOHN W. STORRS,[46] M. AM. SOC. C. E. (by letter).[46a]—This paper is not only interesting and informative, but instructive, and must have required much thought, research, and investigation.

Old wooden bridges have recently been the subject of a number of brochures. Many people are showing an interest in these structures. As an exemplification, there was an unique service at the site of an 82-year old bridge in the Town of Boscawen, N. H., on the afternoon of December 3, 1932. This is a covered bridge of the Burr type with arches, and it spans

[46] Cons. Engr., Concord, N. H.

[46a] Received by the Secretary December 28, 1932.

the Contoocook River. It has been in constant use since 1850. Repairs have been completed recently, and it will do service for a long term of years to come.

The bridge at Woodsville (in the Town of Haverhill, N. H.) built in 1828, and the one in Bath Village built in 1832, are still in service and apparently taking care of the traffic in a satisfactory manner.

The writer would be one of the last to cast any reflections, to find fault with or criticize the earlier American builders of bridges. They were limited as to means and material; had little to go by except trial and experience. Some of the so-called hybrid or mongrel bridges did, or have done, extraordinary and remarkable service, and the constructors and bridge carpenters of those days "did the best they could; angels could do no more."

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

DISTRIBUTION OF SHEAR IN WELDED CONNECTIONS

Discussion

BY MESSRS. F. T. LLEWELLYN, A. S. WOODLE, JR., MILTON MALE, WILLIAM HOVGAARD, CHARLES W. CHASSAING, AND F. E. FAHY

F. T. LLEWELLYN,[2] M. AM. SOC. C. E. (by letter).[2a]—The Engineering Profession is greatly indebted to the author for this valuable contribution toward placing the actual behavior of certain forms of lap-joint on a rational basis. In its major features the analysis now presented is identical with a study which Mr. Troelsch showed to the writer in the summer of 1928, and of which blueprint copies were issued later under date of September, 1928. The writer believes that Mr. Troelsch's method of analysis applies to any form of joint, whether welded, riveted, or otherwise connected, in which the elements that transmit shear are disposed longitudinally. Comment will be given under the heads of mathematics, assumptions, detrusion ratio, and applications.

Mathematics.—Several curious relations, derivable from Mr. Troelsch's expressions, may be pointed out. Regardless of the relation between bar areas, in all cases the shear curve is symmetrical about the origin, although not necessarily of equal extent on both sides. When the area of Bar 1 (Fig. 1) is greater than that of Bar 2 the maximum shear will occur at the end of Bar 1.

Let z_1 be the distance from the origin to the farther end of the joint, and z_2, the distance to the nearer end. Then, for all ratios of bar area and for all values of z, the distance from the origin to the point of average shear lies between z_1 as a maximum, and $\frac{z_2}{\sqrt{3}}$ as a minimum, but never attains either limit.

NOTE.—The paper by Henry W. Troelsch, M. Am. Soc. C. E., was published in November, 1932, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[2] Care, U. S. Steel Corp., New York, N. Y.

[2a] Received by the Secretary November 29, 1932.

Some expressions may be simplified by using functions of $\frac{z}{2b}$ instead of $\frac{z}{b}$. Thus, let v_1 be the shear at one end of the joint and v_2 the shear at the other end. Then,

$$v_1 + v_2 = \frac{P}{N b} \coth \frac{z}{2 b} \qquad (25)$$

Again, let v_m be the shear at mid-length of the joint; then,[3]

$$v_m = \frac{P}{2 N b} \operatorname{cosech} \frac{z}{2 b} \qquad (26)$$

In the special case when $a_1 = a_2$, all the author's expressions may be simplified, as follows:

Equation (15) becomes,

$$v = \frac{P}{2 N b} \cosh \frac{x}{b} \operatorname{cosech} \frac{z}{2 b} \qquad (27)$$

Equation (16) becomes,

$$v_{\min} = \frac{P}{2 N b} \operatorname{cosech} \frac{z}{2 b} \qquad (28)$$

Equation (17) and (18) become,

$$v_1 = v_2 = \frac{P}{2 N b} \coth \frac{z}{2 b} \qquad (29)$$

and Equation (22) becomes,

$$\cosh \frac{x_a}{b} = \frac{2 b}{z} \sinh \frac{z}{2 b} \qquad (30)$$

Assumptions.—The assumptions on which Mr. Troelsch's analysis is based might be acceptable ordinarily. In the present investigation, however, one of them must be qualified in order to represent correctly the actual behavior of joints. Another one must be investigated further if the analysis is to be of practical use.

The assumption that at any section the stress in each bar is distributed uniformly over that bar is not warranted either by the facts as to the bars or by the resulting effect on the welds. A method of determining the variable distribution and of translating it into an equivalent uniform distribution has been presented[4] by W. H. Weiskopf, Assoc. M. Am. Soc. C. E., and Milton Male, Jun. Am. Soc. C. E. Without some such qualification the observed results of certain tests differ from the theoretical values by as much as 30 per cent.

The assumption that there is no tension or compression in the weld metal presumably refers to direct stress lengthwise of the joint. This assumption, although not rigorously correct, is acceptable for the reason that the effect of direct lengthwise stress is apparently negligible as compared with the deformation due to shear.

[4] "Stress Distribution in Side-Welded Joints," by W. H. Weiskopf, Assoc. M. Am. Soc. C. E., and Milton Male, Jun. Am. Soc. C. E., *Journal*, Am. Welding Soc., September and December, 1930.

Mr. Troelsch avoids the necessity of analyzing the internal behavior of the welds by assuming the diagrammatic representation shown in his Fig. 1(*c*), wherein welds of rectangular form are depicted. The internal behavior of such prisms when subjected to shear on opposite sides is readily analyzed. Actually, the welds are of the triangular fillet form indicated in the section to the right hand of the author's Fig. 1(*b*). The internal behavior of a triangular prism subjected to shear on adjacent sides is evidently quite different. Mr. Troelsch makes no attempt to analyze the effect of different forms of weld. He requires that the value of the detrusion ratio, *D,* be determined experimentally for each material and for the cross-sectional dimensions of each form of uniting prism. The need of so doing militates against the predetermination of the manner in which a given joint may be expected to behave. This feature will be discussed in detail later.

Until recently, the assumption that all the elements of the joint deform in accordance with Hooke's law was considered a necessary precedent to analysis. In the discussion as to applications, that will be presented, it will be shown that actual behavior in practice can be reconciled with theory only by assuming that certain portions of the joint do not deform in accordance with Hooke's law. However, the writer is not in position to suggest a practical development of this alternative assumption.

Detrusion Ratio.—The determination and the proper application of the detrusion ratio, *D,* are essential preliminaries to the establishment of any relation between the theory and the results of tests. Evidently, the detrusion ratio is not the shearing modulus of elasticity of the material in question, but it is related to that modulus by some function of the shape and dimensions of the weld. Mr. Troelsch merely defines detrusion ratio as the measure of the stiffness of the weld. This is very true, but how shall its value be ascertained in practice? In a shearing test within proportional limits on a side-welded joint, if the distribution of stress were constant throughout the length of the weld, and if measurement could be made of the relative motion of the critical elements of the weld, its true detrusion ratio would be determinable by dividing the load (in pounds per square inch) by the amount of this relative motion or detrusion (in inches). The extent to which these conditions are attainable will now be discussed.

It is known by theory and confirmed by numerous tests, that the distribution of stress is not constant throughout the length of the weld. Its variation, however, can be averaged by plotting a sufficient number of observed deformations. As Mr. Troelsch points out, their mean value is the detrusion by which the average shear load is to be divided in order to determine the value of *D*. Thus, the actual condition of variable stress would cause no difficulty if the other condition could be met. Unfortunately, no practicable method has yet been suggested for measuring the relative motion of the critical elements of the weld. All the measurements thus far reported have had to be made between points outside the weld, and, therefore, the value of *D* derived therefrom denotes the stiffness of a zone made up of the weld plus strips of adjacent material rather than the stiffness of the weld itself.

This value of D may be called the apparent detrusion ratio. No entirely satisfactory method of relating this ratio with the true detrusion ratio of the weld has yet been presented.

If it were physically possible to eliminate the need of establishing this relation by measuring the deformation of points on the weld itself, the question would still remain as to what points should be selected. In the case of a triangular fillet weld the only accessible points on the legs are at the ends of the hypotenuse. Presumably the deformation between these points would be greater than between any other points on the weld, and, therefore, it would not be an indication of the average behavior of the weld which is evidently the behavior contemplated in Mr. Troelsch's definition of D.

The most important feature that will pave the way toward the removal of these obstacles, and the reconciliation of the theory with all the observed test results, seems to be a rational analysis of the internal behavior of a triangular prism when subjected to shearing forces on its two adjacent legs, and the translation of this behavior into a quantity which, when multiplied by the known modulus of elasticity in shear, or possibly in tension, of the material, will give the true detrusion ratio, D, of the prism. One such analysis is tentatively offered in the paper by Messrs. Weiskopf and Male.[4] As far as the writer is aware, no criticism of this analysis has been presented and no substitute has been offered. A settlement of this feature would be of great value to those interested in the entire problem.

Applications.—All the previous comment has been predicated on the behavior of joints within the proportional limit, in line with one of Mr. Troelsch's fundamental assumptions. Numerous tests have now been made in which care was taken to keep within the elastic limit of the material. In many cases a beautiful correspondence between such tests and Mr. Troelsch's theory has been disclosed. It is proper to inquire what bearing the theory has on actual design, and also to what extent its conclusions are applicable under stresses beyond the elastic limit. The two questions are largely identical for the reason that practically all structures, although designed on the assumption of proportional deformation, involve local concentrations of stress that are far beyond the limits not only of proportionality, but also of elasticity. Engineers have adopted certain conventions which prevent such excess from becoming dangerous. In riveted work, they provide against dangerous concentration of stress on a portion of the rivet shank by restricting the minimum size of rivet that shall be used under specified conditions. It may be that similar legislation is required with respect to the cross-sectional sizes of fillet welds. One fact is quite apparent; namely, that the practical value of analysis such as that presented by Mr. Troelsch is qualitative rather than quantitative. The following example, which illustrates this fact, is interesting because it occurred, not in a laboratory, but in construction of considerable magnitude.

In 1930, during the construction of a new section of subway in New York City, the temporary roadway was carried on 27 and 30-in., rolled-steel girder beams which were spliced, end to end, not at the supports, but at overhanging

points. The splices were made by means of side-welded joints. Computations based on Mr. Troelsch's theory indicate that the end shear was five times the average for which the joint was designed. An over-charge of dynamite caused stresses much beyond the contemplated load. If the welds had behaved proportionately they should have failed under their designed load, whereas they did not fail even under the added effect of the blast. The efficiency of the welds as a whole was doubtless due to their relief by a local straining of the ends beyond the elastic limit. This does not indicate any error in the theory. The example is merely one more illustration of the fact that actual construction involves conditions that are not assumed in any single theory.

A. S. Woodle, Jr.,[4] M. Am. Soc. C. E. (by letter).[4a]—The fundamental assumption on which this paper is based, is that the shearing deformation in the weld between the bars varies from a minimum at some point in the weld to a maximum at each end. The writer is unable to see the reason for this assumption.

It would seem that like most deformations, that of the bars in question would have a definite direction, and that this direction would be in the line of the stress in each bar. The two bars, therefore, deform in opposite directions from zero at one end of the weld to a maximum at the other end.

The increase in unit deformation in each bar would follow a straight-line law, providing only that the bars and the weld between the bars are of uniform cross-section and uniform quality. The unit shear deformation in the weld at any point would be the sum of the unit deformations in the two bars at that point, which are necessarily in opposite directions.

As the locus of the sum of the ordinates of any two straight lines is a straight line also, it follows that the unit shear stress in the weld follows a straight-line law and, in the case of two bars of equal cross-section and the same material, would be a constant throughout the length of the weld.

Milton Male,[5] Jun. Am. Soc. C. E. (by letter)[5a]—The results of the analysis on welded joints as reported by Mr. Troelsch are paralleled in the companion paper by Mr. A. Hrennikoff on the distribution of stresses in riveted joints. The following comments are devoted to Mr. Troelsch's paper.

The "detrusion ratio," D, which is stated to vary with the nature of the weld, and the value of which is to be determined experimentally, may be derived theoretically[6] and its value shown to be a function of the shearing modulus. For the case of a 45° fillet the ratio is equal to $\frac{3}{4}\,G = 8\,700\,000$ lb. per sq. in. In any rational development of a theory of stress distribution in welded joints, the importance of a correct conception of the manner in which

[4] Engr. of Plant, Baldwin Locomotive Works, Philadelphia, Pa.

[4a] Received by the Secretary November 22, 1932.

[5] Care, U. S. Steel Corporation, New York, N. Y.

[5a] Received by the Secretary November 29, 1932.

[6] "Stress Distribution in Side-Welded Joints," by Messrs. Weiskopf and Male, *Journal*, Am. Welding Soc., September, 1930.

the weld acts under shearing deformation warrants further study and experimentation. Indeed, it forms the starting point on which the behavior of the joint as a whole can be predicated.

A graphical illustration of the behavior of fillet welds under the relatively simpler condition of loading—that of an end weld in shear[7]—indicates that the shearing stresses vary along the leg of the fillet by amounts ranging up to 450% of the stress in the bars. It is obvious that for purposes of design some constant intermediate value must be assumed which will approximate the results determined experimentally.

In the analysis by Mr. Troelsch, Equation (15) of the shear curve was derived by assuming that at any transverse section through the bars, the stress carried by those bars was distributed uniformly over their respective total cross-sectional areas, a_1 and a_2. This assumption is one that is commonly made, with a negligible degree of error, in connection with many features of design.

In the case of the welded joints under consideration, however, the error becomes important, because this assumption gives results that vary considerably from many specimens tested. Since Mr. Troelsch developed his equations, many similar equations have been derived independently, but in none but those of Weiskopf,[6] is the deformation in the bars taken into account. In this analysis, checked by many measurements made on specimens tested both at Massachusetts Institute of Technology, Cambridge, Mass., and at the University of Pittsburgh, Pittsburgh, Pa., the stresses in the bars are shown to be anything but uniform, and the parts next to the weld are shown to be stressed to a much greater degree than those farther away. Fig. 5 is an exaggerated representation of a side-welded joint under load, in which, Δ is the deformation of the joint bars, and q_0 and q_z are the deformations at the ends of the welds. In the particular case illustrated, the area of the joint bars exceeds the area of the grip bar; therefore, the maximum deformation q_0, and maximum shear, v_0, occur at the end where the larger area terminates; that is, at the end of the joint bars. To assume that the shear distribution along the weld is the same for both uniform and non-uniform stress in the bars is not correct, and may lead to erroneous results.

It has also been shown[6] that, depending on the ratio of the width of the bar between welds to the length of the weld, the percentage of the total cross-sectional area of the bars that is effective in transmitting stresses into the weld may have some value between 100% and 37%, the percentage of effective area decreasing as the bars increase in width. Curves were plotted, which reduced the laborious computations necessary to determine these effective areas to simple slide-rule calculations.

The author's Examples 1, 2, and 3 indicate that in these specific cases the maximum shears are from three to six times the average shear. Since general design practice dictates the use of a safety factor of 4, it would follow necessarily that the welds should fail under loads far less than the working values, and they would so fail, provided these stress ratios are valid

[7] "Calcul des Soudures prismatiques à Section Triangulaire," Streletsky and Nikolaieff, First Communications of the New International Assoc. for the Testing of Materials, Group D, 1930.

beyond the elastic range assumed in the analysis. The satisfactory behavior of welds when tested is evidence that the stresses tend to re-distribute themselves when the elastic limit is exceeded. In other words, the plastic deformation that the welds undergo by the time the ultimate load is reached, produces a shear curve more nearly approaching a horizontal straight line.

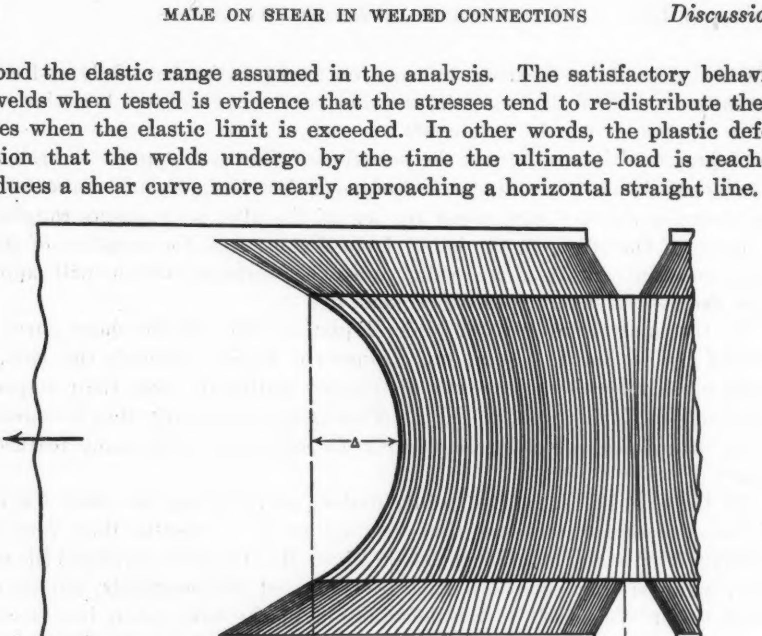


FIG. 5.—EXAGGERATED REPRESENTATION OF A SIDE-WELDED JOINT UNDER LOAD.

What then is the practical value of these analyses? To the writer's mind they serve as a basis for comparing different designs in order to determine

TABLE 1.—COMPARISON BETWEEN THEOTETICAL AND ACTUAL BEHAVIOR OF SIDE-WELDED SPECIMENS

Specimen mark	Number of identical specimens	Length of each weld, in inches	Observed average ultimate strength, in kips per linear inch	Theoretical ratio of maximum to average shear	RELATIVE STRENGTH, PERCENTAGE	
					Theoretical	Observed
2233..........	8	6	9.5	3.0	100.0	100.0
2243..........	8	8	9.2	3.1	96.8	97.2
2253..........	9	10	8.6	3.4	88.2	90.5
2263..........	8	12	8.7	3.35	89.6	91.6
2433..........	8	6	10.1	2.75	100.0	100.0
2443..........	7	8	9.8	3.1	88.8	96.8
2453..........	8	10	9.8	3.1	88.8	96.8
2463..........	8	12	9.5	3.1	88.8	93.7

which is the more efficient, the results being of a qualitative, rather than a quantitative, nature. As an example, a number of side-welded specimens tested by the Structural Steel Welding Committee of the American Welding Society were analyzed to determine what correspondence existed between their theoretical and actual behavior. The results are given in Table 1. It will be observed that although the correspondence between the computed theoretical behavior and the observed test results does not agree in magnitude, these results do show satisfactory correspondence in trend. It may be concluded, therefore, that although the theory does not enable the designer to predict what the actual ultimate strengths of two or more designs will be, it will permit him to determine in advance which of the two will be the more satisfactory in performance.

WILLIAM HOVGAARD,[8] ESQ. (by letter).[8a]—The analysis of shearing stresses in longitudinal fillet welds prescribed by the author for double butt-straps gives the same results as those reported by the writer in 1931.[9] The mode in which Mr. Troelsch's equations have been obtained and the form in which they appear, are different, but they are found to give the same results.

The writer has made many experiments and much extended mathematical analysis in an attempt to obtain the physical constants, notably that called, *D*, in the paper, and also to determine the distribution of stresses in both plates.

The problem is of great importance in ships, where longitudinal girders are often attached in relatively short lengths to a continuous surface, such as a deck, subject to a longitudinal strain. The shearing stresses concentrated at the terminals of such girders have often given trouble, as, for instance, at the corners of deck-houses standing on a strength deck, or at the ends of short longitudinals fitted to the bottom plating of a ship. Whether such connections are riveted or welded, the general stress distribution will be the same; excessive stresses are likely to occur at the ends of the girders.

In Civil Engineering the same problem must occur in many cases, as, for instance, in cover-plates on the chords or flanges of girders. The double butt-strap selected for study by Mr. Troelsch is only an individual case among many.

CHARLES W. CHASSAING,[10] ASSOC. M. AM. SOC. C. E. (by letter)[10a]—The author has analyzed the shear in welded connections by means of a mathematical theory of elasticity. In so doing he has made some curious assumptions and has obtained some strange results.

For example, his assumption that at any section, *f-f*, the stress in each bar is distributed uniformly over that bar, and that there is no tension or

[8] Prof., Naval Constr., Mass. Inst. Tech., Cambridge, Mass.

[8a] Received by the Secretary December 7, 1932.

[9] "The Stress Distribution in Welds"; "The Stress Distribution in Welded Overlapped Joints," *Proceedings*, National Academy of Science, November, 1930; see, also, "The Distribution of Stresses in Welded and Riveted Connections," *Proceedings*, National Academy of Science, June, 1931, and "A New Theory on the Distribution of Shearing Stresses, etc.", *Transactions*, Institution of Naval Architects, London, England, 1931.

[10] Structural Engr., Selden-Breck Constr. Co., St. Louis, Mo.

[10a] Received by the Secretary December 19, 1932.

compression in the weld metal, is in error. It leads to the conclusion that the stress in Bar (1) is not the same as that in Bar (2) at their points of contact. The stresses in each of the bars and in the weld metal must be the same at any point common to each.

In deriving his formulas the author has equated the tension strain to the shear strain. According to the mathematical theory of elasticity the cubical dilatation (that is, the change in volume per unit of volume of a small cube of elastic material in strain) is dependent on the tension and compression stresses entirely. The shearing stresses tend only to distort the cube, but not to change its volume.

The formulas give the greatest unit shear at the ends of the weld where it should be zero. It is impossible to have a shearing stress on the surface of any member that is not in contact with a surface. A shearing stress acting lengthwise with the weld must be accompanied by a shearing stress of equal intensity acting perpendicular to it.

According to the formulas the greatest shearing stress in the weld metal is not dependent on the length of the weld. It is unreasonable to suppose, that by increasing the length of the weld, the unit shearing stress would not be decreased.

The author's analysis does not take into account the dimensions of the weld in cross-section. According to his method, the area of the weld in contact with the bars does not enter into the final results.

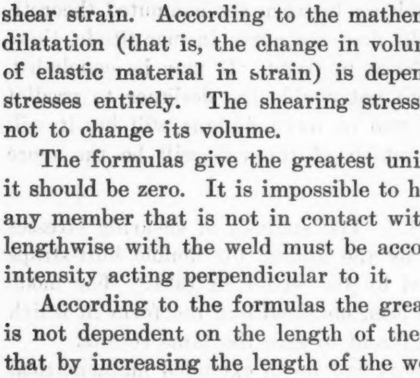


FIG. 6.

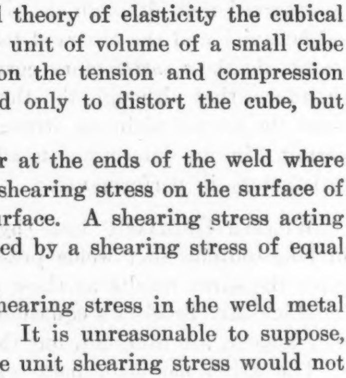


FIG. 7.

In the writer's opinion the problem can best be solved by assuming the weld members to act as a beam at the junction of the members with Bar (2). It is assumed that there is no shear stress on a longitudinal section, taken on the center line of Bar (2) as indicated by the dashed line in Fig. 6. Let Fig. 7 represent a half-section of the weld formed by a horizontal plane as shown. The couple, $\frac{P}{4}\left(\frac{t_1}{2} + t_2\right)$, must be balanced, in order to maintain equilibrium, by a couple, Fl. Consider the force, F, to be the resultants of tension and compression stresses distributed over the longitudinal section of Bar (2). It is scarcely necessary to mention that the other half of Bar (2) will balance these stresses.

There is a moment, $\frac{Pt_1}{8}$, and a shear, $\frac{P}{4}$, acting on each member of the weld. Let w be the width of each member of the weld in contact with Bar (2); z, the length of the weld; and t_1 the thickness of Bar (1). The

moment, $\frac{Pt_1}{8}$, and the shear, $\frac{P}{4}$, will have to be resisted by a beam of width, w, and depth, z. Let f_1 be the unit tension or compression stress at the ends of the weld; then,

$$f_1 = \frac{3}{4}\,\frac{Pt_1}{wz^2} \qquad \text{(31)}$$

The greatest unit shearing stress will occur at the center of the weld and will be one and one-half times the average unit shearing stress. The unit shearing stress at the ends of the weld will be zero. If V_1 is the unit shearing stress at the center of the weld,

$$V_1 = \frac{3}{8}\,\frac{P}{wz} \qquad \text{(32)}$$

Referring to the author's Example 1, consider that Bar (2) is $1\frac{1}{4}$ in. by $6\frac{3}{8}$ in. and that $a_2 = 8.96$ sq in. Furthermore, for Bar (1) (1 in. by 4 in.) let $a_1 = 4$ sq in.; $t = 0.5$ in.; and $z = 6$ in. From Equation (31):

$$f_1 = \frac{3 \times 2 \times 64\,000}{4 \times 6^2} = 1\,333 \text{ lb per sq in.}$$

and, from Equation (32):

$$V_1 = \frac{3 \times 2 \times 64\,000}{8 \times 6} = 8\,000 \text{ lb per sq in.}$$

It seems that the tension stress as well as the shearing stress should be taken into account in designing a welded connection.

F. E. FAHY,[11] ASSOC. M. AM. SOC. C. E. (by letter).[11a]—The author's solution of a rather complicated problem is interesting. It seems worth while, however, to point out that the assumption of a constant value for the detrusion ratio, D, will most likely hold only for short joints under low loads and that the assumption of a uniform distribution of tensile or compressive stresses throughout the cross-section of the connected bars will hold only near the centers of long joints, if at all. That the detrusion ratio, D, will vary in value throughout the length of the joint will be especially true if the bars are thin and, consequently, incapable of much deformation due to shearing forces acting in a longitudinal direction.

Consider two bars fastened together by a layer of glue, weld metal, or other cementing material, as shown in Fig. 8. Let the thickness of this layer of cementing material be called, t, and its shearing modulus of elasticity, E_s. The author's detrusion ratio then becomes equal to $\frac{E_s}{t}$. As t decreases, D increases, and when t is infinitesimally small, D approaches infinity; which means that unless there is a definite slipping or yielding of the cement all of the load, P, must be transferred from one bar to another within an infinitesimally small distance near one end of the joint. Since this is impossible there will be a plastic yielding of the cement at the ends

[11] Asst. Prof., Civ. Eng., Antioch Coll., Yellow Springs, Ohio.

[11a] Received by the Secretary January 9, 1933.

of the joint and a corresponding adjustment in the distribution of the total shearing stress, P, throughout the joint (probably tending toward ultimate uniformity unless there is rupture in the meantime). This assumes, of course, that the bars are drawn up tightly against each other before welding, and neglects the bending effect of eccentricity.

The same argument holds for fillet welds. In Fig. 9 (in which t has been exaggerated for clearness), the capacity of the weld to deform, as shown by

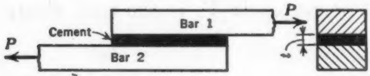


FIG. 8.

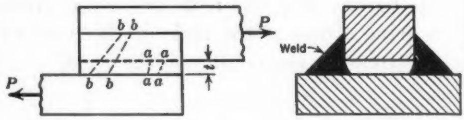


FIG. 9.

the dotted lines, a-a, depends almost entirely on the distance between the bars, assuming perfect fusion throughout. Under this same assumption of perfect fusion the capacity of the weld to deform, as shown by the dotted lines, b-b, depends on the capacity of Bar 1 to deform along those lines, which deformation in the case of a thin bar is quite small.

Even neglecting the foregoing considerations, however, it will be clear from the author's examples that the shearing stresses at the ends of the joint are beyond the range of elastic deformation of the weld metal. In his Example 2 the end shear of 8 040 lb per lin in. of weld is equivalent to 21 400 lb per sq in. at the base of a $\frac{3}{8}$-in. fillet, or 30 300 lb per sq in. of throat area; which means that plastic deformation of the weld metal will occur before the extreme values given by Equations (17) and (18) can be reached. A re-distribution of stress throughout the joint will result, with an ultimate distribution curve lying somewhere between the author's curve and the straight line of average shear.

The fact that the author's assumption of a uniform stress distribution throughout the cross-section of the bars is not entirely true, would tend to invalidate the foregoing comments to some extent, although by no means completely. There can be little doubt that even under low loads the high stress concentrations at the ends of the joint will cause plastic deformations that will result in lower stresses near the ends and higher stresses toward the center than the author's results show. This plastic flow near the ends of the joint means (perhaps fortunately) that the strength of a fairly ductile joint is not entirely limited by high shears at the ends.

A complete mathematical solution, which would take into account the deformations of the connected bars due to longitudinal shears, would be valuable; but it is perhaps impracticable. Photo-elastic tests on celluloid models, or a series of carefully made strain-gauge readings at a number of points along actual welded joints, would be of real value in pursuing, experimentally, the line of reasoning that the author has introduced.

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the *Board* in this manner.

It is especially urged, in communications concerning applicants, that a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from February 15, 1933.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct important work	35 years	12 years*	5 years of important work
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years of important work
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

FOR ADMISSION

ADAMS, LAWRENCE FREDRIC, Newark, N. J. (Age 23.) Instructor, Newark Technical Evening School. Refers to J. E. Beswick, H. N. Cummings, L. G. Cutler, F. P. Gilbert, P. F. Kruse, F. C. Sellnow, A. L. Sherman, M. R. Sherrerd.

AFRICANO, ALFRED, Union City, N. J. (Age 24.) Asst. Engr., Interborough Rapid Transit Co. Refers to O. G. H. Buettner, F. W. Gardiner.

ARNOLD, GERALD EUGENE, San Francisco, Cal. (Age 31). Acting Purification Engr., San Francisco Water Dept. Refers to R. E. Andrews, N. A. Eckert, T. W. Espy, W. B. Freeman, M. C. Hinderlider, G. W. Pracy, F. W. Whiteside.

BISSELL, EARL CHALFANT, Oakland, Cal. (Age 45.) Engr., Port of Stockton, Cal. Refers to K. D. Hauser, W. H. Kirkbride, A. H. Labsap, T. P. Moorehead, F. W. Panhorst, G. W. Rear.

HURLEY, JOHN PATRICK, Nassau, Bahama Islands. (Age 54.) With U. S. Consular Service. Refers to G. Berry, C. B. Galvin, J. P. Hogan, F. Kurtz, T. S. Shepperd, W. W. Stone, W. R. Tenney.

HUTCHISON, ALFRED DRESEL, Marshall, Tex. (Age 34.) Res. Engr., Texas Highway Dept. Refers to W. D. Dockery, G. Gilchrist, G. R. Johnston, C. T. Nitteberg, E. E. Pittman, W. J. Van London, G. G. Wickline.

KINDINGER, MARTIN, Port Elizabeth, South Africa. (Age 36.) Refers to P. R. R. Bisschop, G. F. Huskisson, D. F. Marais, J. Orr, N. Shand, H. A. Smith, W. G. Sutton.

McMANUS, THOMAS KESSLER, Petaluma, Cal. (Age 27.) Mgr., Puget Sound Bridge & Dredging Co. Refers to J. C. Greely, S. H. Hedges, J. Jacobs, H. W. McCurdy, O. A. Piper.

MARDUS, FREDERICK, St. George, S. I., N. Y. (Age 28.) Design Engr. with F. A. Swertz, Cons. Engr., New York City. Refers to W. Allan, C. J. Laube, J. Loewenstein, E. J. Squire, D. M. Zwanziger.

MEYER, JOHN GAVIN, Sacramento, Cal. (Age 30.) Associate Highway Engr., California Div. of Highways. Refers to A. D. Edmonston, A. Givan, S. M. Hands, H. Smitherum, R. H. Wilson, G. Zander.

PALMIERI, MARIO, Sacramento, Cal. (Age 35.) Asst. Bridge Designing Engr., Bridge Dept., California Div. of Highways. Refers to O. R. Bosso, J. Cherno, J. Gallagher, F. W. Panhorst, A. Taylor, W. M. Thomas, E. Weidemann.

SAHNEY, JAGDISH CHANDRA, Jhansi, India. (Age 24.) Civ. Engr. Refers to H. A. Dunlap, F. F. Fergusson, S. K. Gurtu, J. Husband, K. Singh.

SANDERS, WILLIAM LEANDER, JR., Beaumont, Cal. (Age 26.) Jun. Engr., Metropolitan Water Dist. of Southern California. Refers to G. E. Baker, J. B. Bond, R. B. Diemer, F. W. Hough, N. M. Imbertson, S. L. Parratt, W. W. Wyckoff.

STEUBER, WILLIAM FLOYD, La Crosse, Wis. (Age 28.) Res. Engr., Wisconsin Highway Comm. Refers to T. M. Reynolds, M. C. Steuber, E. N. Whitney.

STUBBS, FRANK LYCURGUS, Sabinal, Tex. (Age 26.) Asst. Res. Engr., Texas State Highway Dept. Refers to D. H. Askew, G. G. Edwards, M. B. Hodges, H. S. Kerr, J. E. Pirie, J. G. Rollins.

SUMWALT, ROBERT LLEWELLYN, Columbia, S. C. (Age 37.) Prof. of Civ. Eng., Univ. of South Carolina. Refers to J. B. Babcock, 3d, H. Beebe, J. E. Gibson, T. K. Legare, C. M. Spofford.

THOMPSON, PAUL WILLIAMS, Vicksburg, Miss. (Age 26.) Officer, Corps of Engrs., U. S. Army; Asst. to Director, U. S. Waterways Experiment Station. Refers to G. R. Clemens, W. B. Gregory, H. Hodgman, G. H. Matthes, F. A. Nagler, C. R. Pettis, H. D. Vogel, G. R. Young.

TOERNER, JOHN OSMOND ALOYSIUS, New York City. (Age 33.) Constr. Engr., Manufacturers Trust Co. Refers to F. L. Castleman, J. J. Costa, E. W. Fickes, E. B. Van Hook, J. H. Wickersham, W. H. Yates.

TURNER, LEE, New York City. (Age 38.) With A. M. Byers Co., Pittsburgh, Pa. Refers to G. D. Andrews, C. A. Emerson, Jr., H. M. Freeburn, G. G. Jacobosky, R. L. Sackett, L. Schmidt, D. L. Smith, H. S. Smith, W. L. Stevenson, E. D. Walker.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

FINEREN, WILLIAM WARRICK, Assoc. M., Gainesville, Fla. (Elected Oct. 5, 1909.) (Age 53.) Cons. Civ. Engr., and Asst. Prof. of Mech. Eng., Univ. of Florida. Refers to C. C. Brown, H. D. Mendenhall, J. C. Pinney, C. H. Ruggles, F. T. Williams, G. A. Youngberg.

HARDER, ERNEST HENRY, Assoc. M., East Orange, N. J. (Elected May 13, 1918.) (Age 44.) Senior Bridge Designer, New Jersey State Highway Dept., Newark, N. J. Refers to L. Bush, H. Englander, M. Goodkind, T. H. Irving, C. T. Morris, W. Mueser.

HOERNER, CHARLES GOTTLOB, JR., Assoc. M., Brooklyn, N. Y. (Elected Sept. 10, 1918.) (Age 44.) Sec. Engr., Board of Water Supply, City of New York. Refers to R. W. Armstrong, J. A. Guttridge, G. G. Honness, W. B. Hunter, T. Merriman, J. W. Smith.

JONES, HENRY MACY, Assoc. M., Los Angeles, Cal. (Elected Oct. 15, 1923.) (Age 40.) Secretary, California State Board of Registration for Civil Engineers. Refers to P. H. Albright, D. M. Baker, R. A. Hill, R. J. Reed, A. L. Sonderegger, O. A. Stone, C. R. Sumner.

LARSON, HARRY, Assoc. M., Iowa City, Iowa. (Elected Junior Oct. 21, 1924; Assoc. M. Aug. 30, 1926.) (Age 37.) Fellow, Coll. of Eng., State Univ. of Iowa. Refers to N. W. Bowden, A. H. Fuller, H. A. Hickman, C. T. Johnston, A. Marston, D. W. Mead, R. L. Morrison.

POLLEY, EDWARD RICHARD, Assoc. M., New York City. (Elected Oct. 1, 1926.) (Age 37.) Vice-Pres. and Gen. Mgr., Fairchild Aerial Surveys, Inc. Refers to C. H. Birdseye, C. B. Hawley, T. P. Pendleton, R. H. Randall, S. D. Sarason, G. D. Whitmore.

FROM THE GRADE OF JUNIOR

AXLINE, EDWIN JASPER, Jun., Billings, Mont. (Elected March 5, 1928.) (Age 29.) Project Engr., State Highway Comm. of Montana. Refers to T. V. Bogy, H. Gerharz, J. B. Girand, A. F. Harter, T. A. Munson, R. D. Rader, J. J. Richey.

COLBURN, ROBERT TALBOT, Jun., Passaic, N. J. (Elected April 7, 1924.) (Age 31.) Structural Engr. and Designer, Forstmann Woolen Co. Refer to R. W. Burpee, L. M. Gentleman, H. A. Hageman, L. N. Reeve, G. R. Rich, D. M. Wood.

FRASER, DONALD JOHN, Jun., San Francisco, Cal. (Elected Nov. 14, 1927.) Age 32.) Draftsman, Pacific Gas & Elec. Co. Refers to V. R. Covell, R. G. Hackett, R. F. Krafft, H. V. Lutge, C. M. Mardel.

LOTHES, HERBERT GEORGE, Jun., Cincinnati, Ohio. (Elected Oct. 14, 1929.) (Age 32.) Gen. Supt. of Constr., Kroger Grocery & Baking Co. Refers to J. E. Allan, Jr., R. A. Anderegg, W. W. Carlton, H. B. Luther, R. W. Renn, H. Schneider, C. O. Sherrill.

NORRIS, MILTON SADTLER, Jun., Baltimore, Md. (Elected Dec. 5, 1927.) (Age 32.) Refers to A. C. Clarke, E. J. Dougherty, H. A. Lane, P. G. Lang, Jr., R. H. Lee, R. Mather, J. P. Ray.

STRIEGLER, RICHMOND HOBSON, Jun., San Anselmo, Cal. (Elected Aug. 18, 1930.) (Age 32.) Jun. Civ. Engr., Constr. Quartermaster Dept., War Dept., U. S. Army. Refers to N. Aanonsen, J. E. Allen, O. N. Floyd, G. C. Green, E. H. Hatch, J. L. Lochridge, R. D. Reeve.

The Board of Direction will consider the applications in this list not less than thirty days after the date of issue.